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CONTENTS

Papers and Reports 1-60

Planning for the Postwar Period 1
Report of ACI Committee on Postwar Planning, M. O. WITHEY, Chairman

Charts and a Direct Method for Design of Cantilever Retaining Walls .....
..... WILLIAM A. JONES 5

Supplementary Data on the Effect of Concrete Aggregate Having Low Thermal Coefficient of Expansion ..... J. C. PEARSON 33

Reinforced Concrete Corners in Tension ... C. J. POSEY and ORVILLE KOFOID 41

Chart for the Design and Adjustment of Concrete Mixes ... HOWARD E. BURR 53

Job Problems and Practice 61-76

The Use of Coral Aggregate, Ben E. Nutter

Bond in Isolated Footings, T. D. Mylrea

"Influence of Sands, Cements and Manipulation upon the Resistance of Concrete to Freezing and Thawing", F. V. Reagel

How to Determine Relative Workability, R. F. Blanks, J. C. Pearson, T. C. Powers

Vibratory Remolding Test as a Measure of Concrete Workability, Charles E. Wuerpel

Current Reviews 77-92

News Letter 1-8

Report of 1943 Nominating Committee ● The 40th Annual Convention ● Who's Who ● New Honorary Members ● New Members ● Honor Roll ● Personal ● Obituary

to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge

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DISCUSSION

Discussion closes March 1, 1944

Sept., Jl. '43

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Vol. 15 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN September 1943

Planning for the Postwar Period

A report by the American Concrete Institute's Committee on Postwar Planning

MORTON O. WITHEY, Chairman

Roy W. Crum — Ben Moreell — John R. Nichols
Douglas E. Parsons — Harry F. Thomson — Stanton Walker
Harvey Whipple, Secretary

—A—

Believing that serviceable structures are achieved by planning, engineers welcome the challenge to contribute to plans for the creation of an orderly world from one in conflict and disorder, to help solve the problems of social and economic dislocation produced by war and to assist in the correction of those maladjustments from which wars gather their destructive force. In thinking beyond the sphere of its engineering specialization, there is assumed in American Concrete Institute membership a sense of the responsibility of all citizens for the general welfare—a responsibility made more general and more urgent by the magnitude of the tasks to be accomplished. To these ends it is recommended:

1) That the American Concrete Institute encourage and participate in the discussion of ways and means toward a better postwar world—first, to assist during the emergency of reconversion to peace economy, and second, to contribute to the ultimate goal of domestic and international relations in which an efficient and equitable economy will be general and permanent (Sections B and C).

2) That the American Concrete Institute rededicate itself to the task of increasing, correlating, and disseminating the special engineering knowledge of its chosen field, and of making such knowledge available in readily usuable form for prompt application to the projects of a postwar world; specifically, that:

a) Under a procedure to be inaugurated and directed by the Publications Committee with such special instructions as the Board of Direction may deem best, "Planning for the Postwar Period" be opened to discussion at the Institute's 40th Annual Convention.

b) The Advisory Committee be asked to give prompt consideration to the ACI technical committee activity for the next two years with these objects in view: (1) The inauguration, and prompt completion of assignments to tasks involving reports digesting and correlating needed information in reference to concrete design, construction and manufacture; (2) The completion, as rapidly as possible, of important assignments which will add to the Institute's recommended practices; (3) The encouragement of research undertakings which give promise of valuable contributions to the background of future correlations and recommendations.

c) The text of this report be published to aid the Institute in planning its own work, and in fostering cooperation with other agencies on general problems.

— B —

Inasmuch as the problems of the postwar world are so numerous, so complicated, and so far-reaching that we can hope to solve them only as the best organized thought is brought to bear on their manifold ramifications, it is further recommended:

1) That the Congress of the United States be urged to create at once a non-partisan national planning body of adequately diversified public representation, to gather, consider, and correlate information, to make public specific recommendations and to cooperate with planning agencies of state, county, city governments and of civic, industrial, professional, agricultural, labor, and other organizations, with respect to domestic and international problems, looking toward the crystallization of policies and the promulgation of measures to meet the emergencies of the postwar period and the long-time adjustment of social and economic relations;

2) That subdivisions of government and civic, industrial, professional, agricultural, labor and other organizations, be urged to create and support planning bodies for the purposes of initiating plans and implementing such plans as may be recommended;

3) That among the subjects, proposals and considerations that might come before such planning agencies study be given to (C).

— C —

The country is faced with the emergency problem of employment during the postwar period for: (a) men in uniform at the war's end, (b) government employees in excess of peace-time requirements, (c) employees of government-owned and privately-operated plants producing war material, (d) employees of privately-owned plants now producing war material, that can be reconverted to peace-time production (anticipating that during reconversion there may be an extended period of unemployment). To meet these emergency needs for postwar employment and to build more permanently for the general welfare, there are many possible solutions, among which are the following:

1) Encourage the development of private enterprise by every means consistent with the general good, including review of controls and fiscal policies applied as war-time measures.

2) Encourage all units of industry, large and small, to plan now for their type and scope of operation after the war. Many producers of war goods will return to their former production for civilian needs; others must convert to new lines and new products. Upon the thoroughness of industrial planning now, postwar employment and productivity will depend, especially in view of the prodigious character of postwar expansion required to equal war production.

3) Encourage the complete readiness for use, by private enterprise and by all levels of government, of a true "shelf" of postwar construction projects for which there are authorizations, surveys, plans and specifications, with financing provided:

(a) Let there be special provision for a repair, maintenance, replacement and new work program that emphasizes jobs long deferred by war, and that can be set in motion with the least loss of time at war's end, to tide over employment for the period in which large projects are got under way. By thorough advance planning provide work in reference to public need, avoiding "made" work that might otherwise be hastily improvised and uneconomically executed without attaining maximum public service.

(b) Let long range construction planning for continuing programs and large projects be with reference to public need and employment demand and be reconciled with construction potentialities, the national economy in general, and the wartime backlog of demand for many consumer goods. Consider the overall construction program with a view to maximum employment consistent with construction needs and a balanced economy. With an assumed national income of $110 billion, $14 billion might be spent for construction, of which four or five billion might be for public works. If construction, other than that for war, continues at war's end at $1.5 billion, large possibilities for employment will depend upon the acceleration of the construction program. If this program attains an annual acceleration of $3.8 billion (the war-time peak) an annual construction total of $14 billion would be attained only by the fourth year. In this is further emphasis of the need that a so-called "shelf" of construction projects consist not in projects merely "proposed" but in those advanced to the point of getting under way at once when men return from the military and industrial war-fronts. Assuming that $5,000 for construction hires one man on-site and 1.5 men off-site in shops, mines, transportation, and so forth, for one year, $14 billion would employ 2.8 million men on-site and 4.2 million off-site, 7 million in all of which 2.5 million might be on public works. Although not eliminating unemployment, such a program would be a substantial contribution from the construction industry toward that end.*

4) Consider employment possibilities in dismantling surplus military installations.

5) Plan and schedule demobilization of the armed forces so far as practicable, to return men to civil pursuits as and where employment becomes available—some regions being ready sooner than others, some skills more in demand than others. Such scheduled demobilization might be made feasible by a reversal of the war training program, refitting men in uniform for return to peace-time occupations. The great military training stations and the established educational institutions, both of

*For the figures in (3) reference is made to public releases of the National Resources Planning Board.

which play such important parts in special training for war, might be used for postwar courses in the manual arts, academic subjects, languages, history and so forth, including political economy and the duties and responsibilities of citizens—often less well understood and practiced than the citizen's civil rights.

6) Consider universal military service first for national safety and also in reference to employment problems. One year of such service for boys will not solve the problem of unemployment but will remove some employment competition.

7) Study opportunities for greater coordination of agriculture with industrial and chemical uses of farm products.

8) Apply a lesson of this war in measures to maintain adequate stockpiles of selected raw materials.

9) Seek a solution of continuing problems of social security as related to unemployment and old age benefits.

10) Continue the study of conservation measures for agricultural and forest lands.

11) Encourage the preparation of master plans for cities, and for metropolitan and regional areas by which a more logical arrangement of their development may be laid down, including provisions for adequate housing, streets, parks, facilities, playgrounds, and means of transportation and for the elimination of slums and blighted areas.

12) Effect international agreements for the elimination of financial and legal obstructions to trade between nations.

13) Encourage that expansion of private enterprise necessary for full employment by measures to make the natural resources of the country more easily available. Give thorough study to the best plans not only for conserving natural resources but for their utilization for the public good.

14) Consider the revision of laws and regulations for the greater freedom of commerce between states of the United States, including the most efficient use of all transportation facilities.

15) Study the possibilities of greater safeguards to the public welfare: (a) in patent law revision that, while giving incentive to invention, would prevent undue delays in the use of new ideas and processes; (b) in such revision of anti-trust laws that, without sacrifice of essential safeguards against uneconomic and monopolistic practices, would encourage efficient production and distribution; (c) in the enactment of laws to discourage the restriction of production for the maintenance of high profit margins; (d) through the cooperation of labor in the elimination of practices that increase costs of production and construction without compensating contributions to the general welfare of labor and the public.

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Charts and a Direct Method for Design of Cantilever Retaining Walls*

By WILLIAM A. JONES†
Member American Concrete Institute

SYNOPSIS

This paper describes and illustrates a method for the design of retaining walls with special reference to the highway field. Included is a suggested classification of walls. It touches lightly upon the selection of wall types and of earth pressure formulas and factors, and discusses and illustrates briefly the preparation of charts.

Both method and charts were developed and used by the author in a large bridge designing office and for the design of practically all wing walls of the numerous bridges of the new Pennsylvania Turnpike, thereby saving design cost and facilitating the intense design pace of the project.

INTRODUCTION

There is need for charts permitting easy and quick design of cantilever retaining walls.

In studying this need, consideration must be given at the very outset by each design office, to the various preliminary assumptions that accompany the design of any retaining wall: weights of earth and masonry, coefficients of friction, type and magnitude of superloads, use limitations, etc., and, conservatively safe values must be assigned to each of these variables. The study of these preliminary assumptions is not included in the scope of this paper.

Having established these values, a method of design that will lend itself to preparation of charts must be developed. The method should be direct; it should necessitate no further assumptions and should arrive at the answer that will fulfill the requirements most economically.

The subject is presented here primarily from the standpoint of the highway field or its equivalent as in that field design charts are probably

*Received by the Institute, Jan. 4, 1943.
†Stone & Webster Engineering Corp., Euclid, Ohio

most needed. In the building field and for railroad and highway bridge abutments, the additional superstructure loads encountered usually vary to such an extent that charts are impracticable. The method described in this paper however is general and will apply to all cases, bearing in mind, of course, that the resultant force referred to therein must first be combined with the additional superstructure loads. The reader will recognize throughout the paper details that are not new. These have been added to avoid any gap in the various design steps.

DESIGN METHOD—STEM AND FOOTING

Stem Design

Top thickness—Theoretically, the top thickness of the stem can be zero. Practically it must be sufficient to provide for pouring facility and strength to withstand impact and frost action. Pouring facility usually governs. For the purposes of this paper (walls up to 35 and 40 ft. high) 11 in. has been considered the minimum clear pouring width advisable and this width plus a 4-in. reinforcement cover at the rear face and 3-in. at the front face, gives a top thickness of 18 in. For lower walls of say 25 ft. and 15 ft. maximum height, this thickness might be reduced to 15 in. and 12 in. respectively. In each case, the design procedure remains exactly the same.

Height limit—35 ft. has been set as the upper height limit and, inasmuch as the upper height limit of economy for this type of wall is usually less than this height, this limit will embrace the majority of cantilever walls. To accommodate unusual and extreme cases, however, stem design charts have been carried up to 40 ft.

W and M_s^W—W is the total weight of the stem and retained material left of the vertical line f-f in Fig. 1.

M_s^W is the moment of W about point s, the point where the reinforcement cuts the base.

The derivation of expressions for W and M_s^W is tabulated:

Part	Computation	Weight	Arm	Moment About Point s
I	$1.50h150$	$225h$	$d - 0.75$	$225hd - 169h$
II	$(d - 1.17)h\frac{1}{2}\ 150$	$75hd - 88h$	$.667d - 1.11$	$50hd^2 - 142.2hd + 97.7h$
III	$(d - 1.17)h\frac{1}{2}\ 100$	$50hd - 58h$	$.333d - 0.72$	$16.7hd^2 - 55.3hd + 41.8h$
IV	$(d - 1.17)(2)\ 100$	$200d - 234$	$.50d - 0.92$	$100d^2 - 302d + 216$

Summation for parts I to IV inclusive:

$$W = 125\,hd + 79h + 200d - 234 \quad \text{(a)}$$

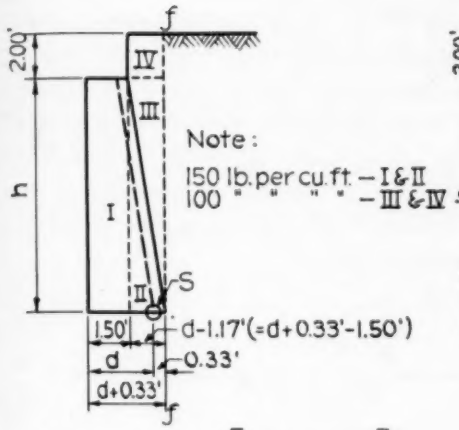


Fig. 1 (above)—$\left[W \text{ and } M_s^W \right]$

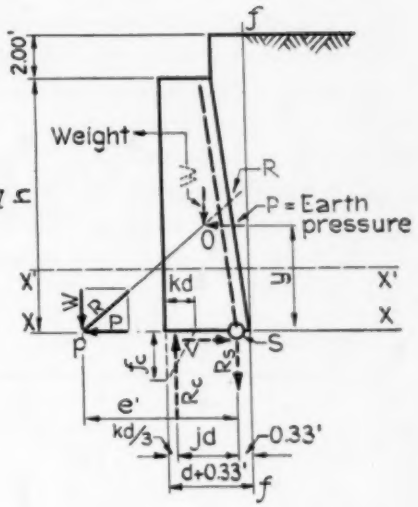


Fig. 2—(right) Forces

$$M_s^W = 66.67\ hd^2 + 27.5hd - 29.5h + (100d^2 - 302d + 216)$$

The terms in parentheses are never more than ⅓ of one per cent of the total moment acting on the structure; so, neglecting these terms:

$$M_s^W = 66.67hd^2 + 27.5hd - 29.5h \quad \text{.....(b)}$$

Bottom thickness—The bottom thickness is determined by applying the laws of statics and flexure to the forces acting on a horizontal section x-x at the base. (See Fig. 2.)

Study of Fig. 2 shows that the external force on section x-x may be regarded simply as a single force R sloping downward and cutting the base at point p, a distance e' from the reinforcement (point s). The moment of this force about point s, $M_s^R = We' = M_s^W + Py$ in which M_s^W is the moment about point s of the vertical component W acting at the center of gravity of the combined concrete and earth sections left of line f-f (i.e. through point o), and Py is the moment about the same point of the horizontal component P acting a distance y above the base.

$$M_s^R = We' = M_s^W + Py \quad \text{.....(c)}$$

This external force R acting on the horizontal section x-x (Fig. 2) is held in equilibrium by three internal forces:

(1) The compressive stress in the concrete R_c acting vertically upward (resisting the tendency of the front part of the wall to move downward).

(2) The tensile stress in the reinforcement R_s acting vertically downward, neglecting the batter, (resisting the tendency of the rear part of the wall to move upward).

(3) A horizontal stress in the concrete, V, acting horizontally to the right (resisting the tendency of the wall to move bodily to the left).

This force system has been isolated in Fig. 3.

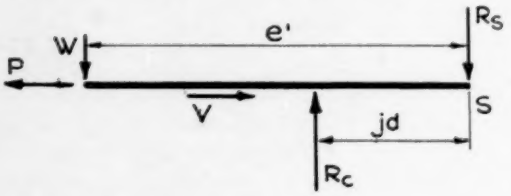


Fig. 3—Design of base

In Fig. 3 note that:

$R_c = \frac{1}{2} f_c kbd \qquad R_s = f_s pbd \qquad V = v\,b\,j\,d$

$W = 125\,hd + 79h + 200d - 234$

$We' = 66.67\,hd^2 + 27.5hd - 29.5h = M_s^W + Py.$

To obtain an expression for d, impose $\Sigma M = O$ about point s, i.e. $We' = R_c jd$; substitute for R_c and We' their values as given by the equations above; substitute the usual constant K for $\frac{1}{2} f_c kj$ and solve for d. The resulting expression, after compensating fully for two minor terms in the answer by introducing the term $+0.02$ ft. is as follows:

$$d = \sqrt{\frac{Py - 29.5h}{12Kb - 66.67h}} + .02\ (\text{ft.}) \qquad \text{(d)}$$

Py may be taken from whatever standard tables are in use in the office, or computed from earth pressure formulae based on theories of either Rankine, Coulomb, Boussinessque, Terzhagi, Spangler, Hogentoger, etc., or any other theory that experience has proven conservatively safe.

The following formulae are Rankine's for a retained material weighing 105 lb. per cu. ft.; a uniform level surcharge h', and an angle of repose of 33 deg. 42 min. They are given here for *no other reason* than that of convenience in establishing the values in the examples referred to below:

$$P = 15\,(h + 2h') \qquad \text{(e)}$$

$$y = \frac{h^2 + 3hh'}{3(h + 2h')} \qquad \text{(f)}$$

To find d substitute the foregoing values in equation (d).
For instance for $f_c = 1000$, $f_s = 18000$, $n = 12$, and $h = 30$ ft.

$k = 0.400$, and $j = 0.867$, $K = 173$
$P = 15300$ lb., $Py = 162000$ ft. lb.
$12Kb = 12 \times 173 \times 12 = 24900$
$66.67h = 66.67 \times 30 = 2000$
$29.5h = 29.5 \times 30 = 885$

$$d = \sqrt{\frac{162000 - 885}{24900 - 2000}} + .02 = 2.67 \text{ ft.}$$

Reinforcement at base—Again, in Fig. 3, imposing $\Sigma V = 0$:

$$R_s = R_c - W$$

Substituting for R_s and R_c, their value given above under Fig. 3, and solving for p:

$$p = \frac{\frac{1}{2} f_c kbd - W}{f_s bd} \quad \text{.....(g)}$$

Thus for the foregoing example:
$W = 125\,hd + 79h + 200d - 234$ (Eq. (a) above) =
$125 \times 30 \times 2.67 + 79 \times 30 + 200 \times 2.67 - 234 = 12670$
$\frac{1}{2} f_c kbd = \frac{1}{2} \times 1000 \times 0.400 \times 12 \times 2.67 \times 12 = 77000$
$f_s bd = 18000 \times 12 \times 2.67 \times 12 = 6{,}950{,}000$

$$p = \frac{77000 - 12670}{6950000} = .00926$$

$As = pbd = .00926 \times 12 \times 2.67 \times 12 = 3.56$ sq. in.

Shear, diagonal tension and bond—In Fig. 2, imposing $\Sigma H = 0$ at the base, the horizontal internal force in the concrete equals P and this value might be taken as the value of the external horizontal shearing force acting on that section. However, there is also acting vertically on that section the entire weight W, and before this horizontal shear could be developed, it would be necessary first to overcome the weight W. Moreover, compared to the horizontal force P, the vertical weight W is of considerable magnitude. Thus for the problem given above, it is 12670 lb., while the horizontal P value is 15300 lb. Therefore, the induced compression in the stem, analogous to that in an arch rib, will not permit the external shearing force to become fully effective. However, even under the assumption that the entire force P were unhampered in producing shearing stresses, shear would rarely govern the amount of reinforcement.

Diagonal tension and bond stresses depend upon external shear, hence the above remarks are also applicable to these stresses.

Design of any section above base—Using a straight rear face batter to the top, (Fig. 2), the thickness of any section x′-x′ above the base is computed by simple proportion. The thickness thus computed will be greater than that needed for balanced concrete and steel stresses because

as the height decreases the external moment decreases as the cube of the height whereas the resisting moment of the stem itself decreases only as the square. Hence the design of any such section becomes merely a question of determining the amount of reinforcement needed and of computing other stresses merely for record. This may be done either by using Beards' Diagrams* or by direct computation, viz,

Using Beards' Diagrams—Compute d by proportion and in turn W (Eq. (a)); compute also M_s^W (Eq. (b)); Py; $We' =$ Beards' $Ne' = M_s^W + Py$;

$$\text{Beards' } K = \frac{We'}{bd^2}; \text{ and } \frac{e'}{d} = \frac{We'}{Wd}$$

Enter Beards' Diagram with K value at bottom left; follow upward to intersect f_s curve and record f_c, thence horizontally to right to intersect the $\frac{e'}{d}$ curve, thence downward to bottom right and read p. Finally compute $As = pbd$.

By direct computation: (1) Compute d by proportion; also W (Eq. (a)); M_s^W (Eq. (b)); Py; $We' = M_s^W + Py$ and $L = \frac{We'n}{bd^2f_s}$. All these are given constants.

(2) Compute k i.e., $k = \sqrt{2.4L + 1.4\,L^2} - 1.18\,L$; f_c from $\frac{kf_s}{n\,(1-k)}$ using the maximum allowable value for f_s; p from (Eq. (g)), i.e., $p = \frac{\frac{1}{2}f_ckbd - W}{f_sbd}$ and finally As pbd.

Shear, diagonal tesnion, and bond—any section above base—The comments concerning shear, diagonal tension, and bond stresses at a section at the base of the wall will apply here with even greater force, for as the height decreases the shear P decreases as the square of the height but the resistance to shear decreases only as the first power.

Two rear face batters—For long high walls it may be economical to use two rear face batters starting, for example, with say a 3-in. batter at the base of the stem and changing to say 2 in. per ft. after the height has sufficiently decreased. These batters and also the height at which the change is permissible may be readily determined from the chart entitled Rear Face Batters in this paper. The additional cost of reinforcement and form work must be considered before deciding to use two batters and consideration given to the fact that the single battered wall, being more massive, would have slightly greater inherent stability.

*See Bending & Direct Stress Diagrams, Case III, in Hool & Johnson's Concrete Engineers' Handbook.

Temperature and shrinkage reinforcement—The 1940 Joint Committee Report, page 91 (d), specifies:

Longitudinal reinforcement for volume changes due to shrinkage and to changes in temperature should be provided in all reinforced concrete walls near the exposed face to an amount of not less than 0.50 sq. in. area per ft. of height with a maximum spacing of bars of 12 in. center to center. It is desirable to use bars of small section and close spacing for this purpose.

The 1941 American Association of State Highway Officials Standard Specifications, page 155, specifies:

Except in gravity walls, not less than ⅛ square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

Note in each case above the requirements are *minimum* requirements. They are equivalent, respectively, to ½-in. diameter bars spaced 4¾ in. and 19 in. center to center.

The former spacing appears to the writer somewhat close, but may have been necessitated by extreme temperature variations, thin sections, rich concrete, spacing of expansion joints, etc.

The amount of reinforcing necessary over and above the minimum specified will depend upon the judgment used in evaluating the relative severity of temperature range, horizontal flexural stresses due to earth pressure variations or differential temperature, continuity stresses at corners, effects of thin sections and rich mixes, etc.

For walls of the general character described herein and for Pennsylvania climate, ½-in. diameter horizontal bars 12 in. center to center along the front exposed face and ½-in diameter, 12 to 18 in. center to center along the rear face has been found entirely satisfactory. The latter bars will serve also as spacers for the vertical rear face bars. In both cases these bars should be wired to the front face side of the vertical bars.

Expansion joints—For walls of the general proportions described herein and for Pennsylvania climate, provide ½-in. tongue and groove flashed expansion joints 60 ft. center to center maximum with no reinforcement passing through the joint. At the center line between expansion joints provide a vertical construction joint with a V notch at the front face, with reinforcement carried through the joint.

Note—The designer should be on the alert for the necessity of joints other than those required for temperature and shrinkage, i.e. to dampen vibration, break continuity, separate heavy masses from thin masses, etc.

Drainage—It is imperative that the backfill be well drained.

Unless very severe surface drainage conditions exist, the following method is recommended:

Provide an 18 x 18-in. section of broken stone backfill extending horizontally for the full length of the wall, sloping 1 in. per ft. towards 4-in.

minimum diameter weep holes spaced 12 to 15 ft. center to center at such elevation that drainage water may readily escape. At each center line between weep holes provide a 12 x 12 in. vertical column of broken stone extending upward to the coping. In the event the backfill below the 18 x 18-in. section is relatively porous, provide a 6-in. bed of dense backfill material immediately under this section. At each weep hole provide a minimum of ⅓ cu. yd. of broken stone.

If very severe surface drainage conditions exist, provide an 18-in. thickness of broken stone over the entire rear face of the wall draining to a perforated or open joint pipe at such elevation that the drainage water may readily escape, and provide suitable access manholes.

Footing Design

Footing thickness—From the standpoint of structural design theory alone the footing thickness at the heel and toe is determined by the allowable concrete stresses or by the limiting spacing or amount of reinforcement. However, the underground hazards often accompanying the pouring of the footing makes it advisable to establish a thickness sufficient to avoid heavily reinforced sections that would be difficult to pour. Thus the footing thickness should be sufficient to make stirrups unnecessary and to make unnecessary also the spacing of heavy reinforcing bars closer than, say, 5 in. c. to c. Moreover, in walls of variable height, up to a height of 35 ft. or more, a thickness sufficient to eliminate frequent changes in thickness is advisable. Furthermore to provide adequate protection for the bottom reinforcement ample cover, say up to 6 in., is advisable. With these considerations in mind the minimum footing thickness has been established tentatively for the purpose of this paper as 3 ft. 0 in. for walls on earth foundation (Types 1, 3 and 4—see below) and 3 ft. 6 in. for walls on rock foundation (Type 2—see below). For walls of lower height range this thickness could be correspondingly reduced to say a lower limit of 16 in.

These thicknesses may be used in investigating stability, designing the width of the footing, and its position with respect to the stem. Any change in thickness later found necessary to meet flexural requirements of the toe and heel will affect stability factors by merely the sectional area of the difference involved times the *difference* between the unit weight of earth fill and concrete. This effect is entirely negligible, and moreover is on the safe side.

W and M_h^W —*W* is the total weight in pounds of the footing, stem, and retained material left of the line f-f in Fig. 4.

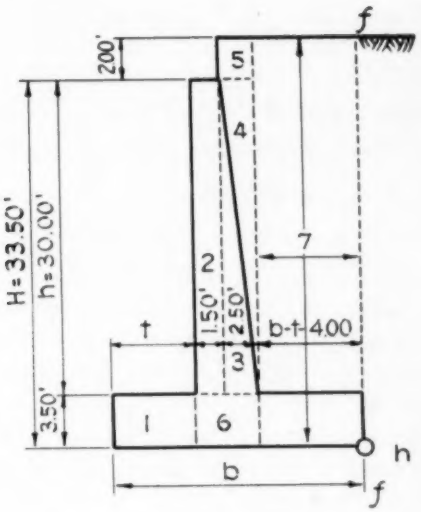


Fig. 4 — W and M_h^W

M_h^W is the moment of W about the heel point h. The derivation of expressions for W and M_h^W is tabulated below:

Part	Weight	Arm	Moment About Heel Point h	
1	$525t$	$\left(b - \frac{t}{2}\right)$	$525bt$	$263t^2$
2	6750	$b-t-.75$	$6750b-6750t$	-5062
3	5625	$b-t-2.33$	$5625b-5625t$	-13125
4	3750	$b-t-3.17$	$3750b-3750t$	-11875
5	500	$b-t-2.75$	$500b-500t$	-1375
6	2100	$b-t-2.00$	$2100b-2100t$	-4200
7	$3725b$ $-3725t$ -14900	$\frac{1}{2}(b-t-4.00)$	$1863b^2-3725bt-14900b+14900t+1863t^2+29800$	

Summation for Parts 1 to 7 inclusive:

$$W = 3725b - 3200t + 3825 \quad \text{(A)}$$

$$M_h^W = 1863b^2 - 3200bt + 3825b - 3825t + 1600t^2 - 5837 \quad \text{(B)}$$

The derivation of the expression for W is comparatively simple and rapid. Even greater ease and rapidity can be had, if desired, by noticing that for the given conditions the coefficients of b and t are constant, that the value of the constant, (in this case 3825) is merely a function of the stem height h, and that a simple study of the elements that determine it will prove it equal to $10.42h^2 - 115.08h - 2112$.

The expression for M_h^W can also be derived easily and rapidly by noting that all the coefficients can be obtained mentally by glancing at the expression for the weight W. Thus:

The b^2 coefficient is one-half the b coefficient in the W expression.

The bt coefficient is equal to the t coefficient.

The b and t coefficients are equal to the constant.

There remains to be determined only the constant (5837) and a simple analysis of the individual elements that determine this constant will show that it is merely a function of h and s, s being the total horizontal projection of the battered rear face. It will be found to be a deductive quantity equal to $h\ (565 + 37.5s + 8.33s^2) - 225$. Had the footing thickness been 3 ft. 0 in. instead of 3 ft. 6 in., this expression would have remained practically the same, the change being merely that the constant 225 would become 226.

These relations hold true regardless of height.

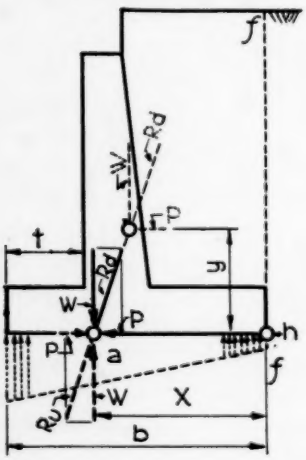


Fig. 5—Forces

Forces and stability—Study of Fig. 5 shows that the external force acting on the footing may be regarded as simply a single force Rd sloping downward and cutting the base at a, a distance x from the heel point h.

The value of the vertical component of this single downward force Rd is equal to the value of the weight W referred to above, and the horizontal component is equal in value to the earth pressure P. It is resisted by an equal and opposite force Ru with vertical and horizontal components equal in value to W and P respectively and passing through point a. Note the value of W is equal to the area of the footing earth

pressure diagram indicated by the fine dotted lines in Fig. 5, and the value of P is equal to the friction developed between the structure and the foundation material.

The downward force Rd referred to above, acting along its line of action oa, tends to lower the stability of the structure by causing:

(1) the foundation material to *crush or settle* excessively or non-uniformly, or both excessively and non-uniformly,

(2) the structure to slide bodily to the left along the foundation,

(3) the structure to overturn about the point of the toe or about a small area of the base adjacent to the point of the toe.

Basis of design for stability—The width of the footing and its position with respect to the stem must be such that the resisting force Ru will be of the proper magnitude, slope, and location, to provide a definite margin of safety against the crushing, sliding, and overturning tendencies mentioned under (1), (2), and (3) above.

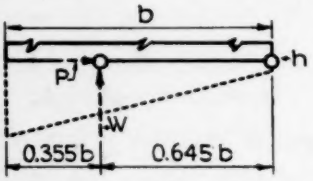
General procedure—design for stability—The first step in determining the width and relative position of the footing to meet stability requirements is to derive expressions in terms of b, the base width, and t, the toe dimension, showing the relation these two variables must bear to each other in order to meet each particular requirement. The second and final step is to determine which pair of values of b and t will safely and most economically fulfill all stability requirements.

Note from either Fig. 4 or 5 that the heel dimension equals the base width less toe dimension and stem width.

Wall types (based on "crushing" requirements)—To provide against undue crushing of the foundation material, the degree and manner in which the earth pressure is concentrated on the toe, as shown by the earth pressure diagram must first be such that the safe bearing value of the foundation material will not be exceeded, and secondly, such as to limit differential settlement to an amount that will not cause the wall to tip excessively. From this standpoint soils may be classified into groups according to safe bearing value and susceptibility to differential settlement and there may then be established for each group a type of wall defined by the shape of the earth pressure diagram—the diagrams being so chosen that for walls up to 35 ft. high the earth pressure will not exceed the safe bearing value for each class, nor cause excessive differential settlement. Thus from a study of 35 ft. walls with say a 2-ft. uniform Rankine surcharge, the following types will be established:

Note—Although "crushing" requirements have been chosen to serve as a basis for determining wall types, it shall not be inferred that "crushing" will be the determining

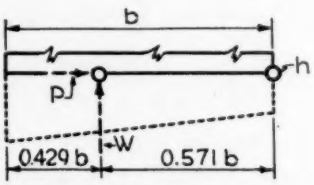
factor for the value of b. This requirement has been considered first merely because it embodies a convenient basis for classifying walls.



ΣM_h:

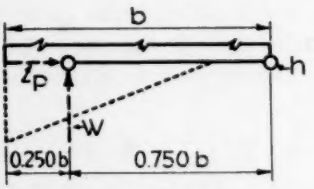
$0.645\,bW = M_h^W + Py$

Fig. 6—Type 1 Walls—0-3 tons per sq. ft. (earth foundation)



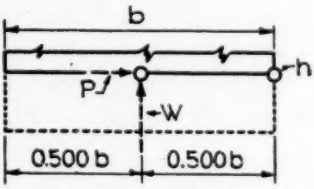
$0.571\,bW = M_h^W + Py$

Fig. 8—Type 3 Walls—0-2 tons per sq. ft. (earth foundation)



$0.750\,bW = M_h^W + Py$

Fig. 7—Type 2 walls—0-5 tons per sq. ft. (rock foundation)



$0.500\,bW = M_h^W + Py$

Fig. 9—Type 4 walls—0-1 ton per sq. ft. (earth foundation)

$$0.645bW = M_h^W + Py \quad \text{(C)}$$

$$0.75bW = M_h^W + Py \quad \text{(D)}$$

$$0.571bW = M_h^W + Py \quad \text{(E)}$$

$$0.500bW = M_h^W + Py \quad \text{(F)}$$

General note for Fig. 6, 7, 8, 9 above—The point marked h is the heel point of the footing.

Concerning pressure diagrams (Fig. 6, 7, 8, 9)—(a) Note above under Type 2 Walls, (Rock Foundation) the resultant falls at the quarter point so that one-fourth of the base is inactive in transmitting pressure and serves only to provide sufficient weight to prevent the resultant from falling beyond the quarter point.

(b) In establishing the pressure diagrams the quarter point has been chosen as the limit of approach of resultant to toe because beyond the quarter point resistance to overturning diminishes very rapidly. In this connection, note that should the resultant fall at the toe, the moment of

resistance of the wall would then just equal the overturning moment and the margin of safety against overturning would be zero. For this reason 5 tons per sq. ft. is the maximum pressure developed by Rock Foundation types.

(c) Type 4 walls will probably seldom be necessary as soils "good" for only 1 ton per sq. ft. will usually call for a pile foundation.

"Crushing" design—Equations (C), (D), (E), and (F), Fig. 6, 7, 8, and 9 respectively, express the relation necessary between b and t to satisfy the crushing requirement and are derived by imposing $\Sigma M = 0$ about the heel point h.

For any specific case, such as the case of 33.50 ft. wall shown in Fig. 4 the values of W and M_h^W given in Equations (A) and (B) can be substituted in Equation (C), (D), (E) or (F), depending on the wall type and the equation solved for b. Assuming in this example that the wall is to bear on a rock foundation capable of supporting safely 10 tons per sq. ft. a Type 2 wall would be selected and the resulting equation for b would be:

$$b = \sqrt{1.90t^2 - 4.56t + 221} - .43t + .52$$

The graph of this equation is shown in Fig. 10. The value of t that makes b a minimum can be found at a glance; or it may be determined directly without a graph by placing the first differential of b with respect to t equal to zero. For this case the values are $t = 4.70$ ft. and $b = 14.0$ ft. These then, are the values consistent with a pressure diagram of the shape required for Type 2 walls, of a maximum pressure intensity less

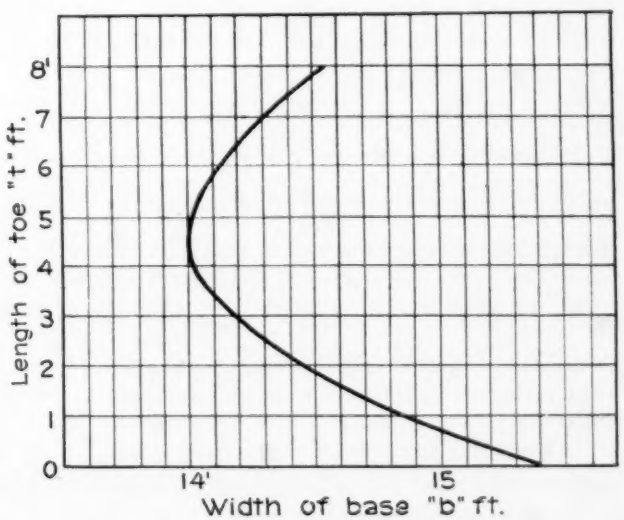


Fig. 10
Crushing graph

than the safe soil bearing value, and of minimum base width for these conditions.

The procedure for any other Type of wall or pressure diagram is exactly the same.

Sliding—The entire structure will slide bodily if the horizontal pressure is able to overcome the total friction that can be developed by the entire weight W against the foundation material. This frictional force is the product of the coefficient of friction and the weight W. For the purpose of this paper a coefficient of friction of 0.570 will be used for rock and 0.417 for earth. A definite factor of safety will be applied by multiplying the pressure to be resisted P by 1.25. Thus for rock (Type 2 walls) the frictional force 0.57 W must equal 1.25 P. Solving for W:

$$W = 2.2P \quad \text{(G)}$$

and for earth (Types 1, 3 and 4) $0.417W$ must equal $1.25P$. Solving for W:

$$W = 3.0P \quad \text{(H)}$$

For the specific case referred to above (33.50 ft. Type 2 wall on rock foundation (Fig. 4) substitution in equation (A) of the t and b values found above under crushing will give $W = 40900$ lb., i.e. 2.27 times the value of P. A weight of $2.2P$ (Eq. G) only is required for sliding, hence sliding does not govern.

However, in order to study a case for which sliding does govern—let us suppose that this 33.5 ft. wall had been a Type 1 wall designed to bear on an *earth* foundation instead of rock and let us follow through an entirely new design from the beginning. For such a case the expression for W would vary slightly due to the use of a 3.00 ft. instead of a 3.50 ft. footing and would be as follows:

$$W = 3700\,b - 3250\,t + 3920$$

As previously explained, with the expression for W established, the value of M_h^W can then, with the exception of the constant 6100, be written down at the stroke of a pencil and is as follows:

$$M_h^W = 1850\,b^2 - 3250\,bt + 3920\,b - 3920\,t + 1625t^2 - 6100$$

Note:—Here again the value, 6100 depends merely on the stem height h and the total width of the batter s and an equation for this value can be derived by adding the general expression for the constants of each component section, the result being as follows:

$$h\,(57 + 37.5s + 8.33\,s^2) - 226, \text{ (deductable)}$$

and is applicable to all walls of this type. Substituting $h = 30.50$ and $s = 2.54$ in this equation gives 6100, used above.

By substituting these values of W and M_A^W in the general crushing Equation (C) and solving for b, the following expression for b is obtained:

$$b = \sqrt{4.18t^2 - 10.1t + 384} - 1.07t + 1.30 \quad \text{(J)}$$

By equating the first differential of b with respect to t, to zero or by inspection from its graph the values b and t that makes b a minimum are found to be $b = 16.5$ ft. $t = 7.06$ ft.

The weight obtained directly from the above equation for W is as follows:

$$3700 \times 16.5 - 3250 \times 7.06 - 13920 = 42000 \text{ lb.}$$

In investigating sliding it is then found that the ratio between 42000 and the earth pressure P is $\frac{42000}{17960} = 2.34$ whereas $3.00\ P$ is required. For this case the b and t values corresponding to a minimum width base for "crushing" will *not* fulfill the sliding requirement. The relation between b and t to fulfill sliding requirements is very simply derived from the fact that W must equal $3.0P$ (Eq. H), substituting for W its value $37.00b - 3250t + 3920$, and solving for b. The result is:

$$b = 13.5 + 0.878\,t \quad \text{(K)}$$

The graph of Equation (K) is shown in Fig. 11.

This equation solved simultaneously or graphically with the crushing equation (J) will give:

$b = 17.20$ ft. and $t = 4.20$ ft.

These then are the values that fulfill both the crushing and sliding requirement and furthermore they are the only pair of values that can exactly fulfill these requirements.

The above procedure is general and has purposely been given in great detail. However, for a series of walls of varying heights, preliminary study of limiting cases would show that for walls up to 35 ft. high defined above as Type 1 walls sliding would always govern and for Type 2 the minimum length base for crushing would govern. This knowledge would allow the designer to solve directly for the governing requirement and to merely investigate the other requirements for record.

Overturning moment—If a wall were supported on the very soundest non-yielding rock, the earth pressure would tend to overturn the wall about the very point of the toe. However, for Types 1 and 3 walls supported on earth and with relatively large pressure intensity at the toe, the earth under the toe will yield and settle slightly so that the point or area about which the wall tips will tend to move inward towards the center of gravity of the combined mass. Furthermore, due to the extreme varia-

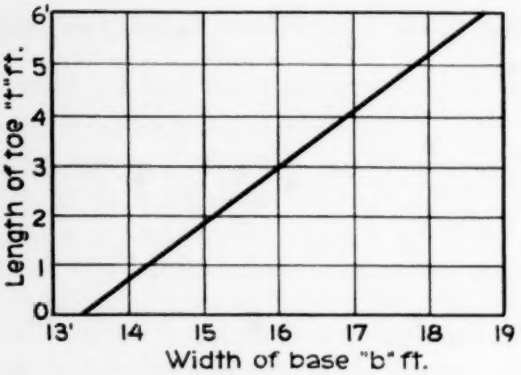


Fig. 11—Sliding graph

tion of pressure intensity (varying to practically zero at the heel) the entire base will tend to yield or settle non-uniformly. The wall will tend to tip forward slightly due alone to the manner in which the earth pressure intensity below its base is distributed. This action may be progressive and tend to decrease the horizontal distance between the pivot and the center of gravity of the combined mass, i.e., to decrease the moment arm and in turn the moment of resistance of the wall itself. In contrast thereto the arm of the earth pressure overturning moment y would tend to increase slightly. To compensate for this tendency the overturning moment Py will be increased 40 per cent for walls on earth foundation (Types 1 and 3), and 28 per cent for walls on rock (Type 2). In addition, a definite margin of safety will be provided in each case by multiplying these factors by 1.25.

Thus walls on earth foundation will be designed to withstand an earth pressure moment of:

$$Py \times 1.40 \times 1.25 = 1.75\ Py, \text{ (Types 1 and 3)}$$

and walls on rock foundation: $Py \times 1.28 \times 1.25 = 1.60\ Py$ (Type 2)

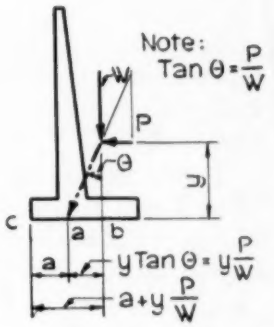
For Type 4 walls (uniform pressure) the first factor may be decreased to say 1.12 since in that case the uniform pressure would tend to produce uniform settlement. The minimum allowable moment of resistance for that case would thus be:

$$Py \times 1.12 \times 1.25 = 1.40\ Py \text{ (Type 4)}$$

Note: As every designer knows, unless an extremely high margin of safety is required, overturning will not usually be troublesome since the resultant must usually fall well within the base to meet other requirements. Every design, however, should provide a definite margin of safety and the actual margin provided by the final design should be recorded.

Moment of resistance to overturning—An expression for the resisting moment a wall offers against overturning can be obtained in the usual manner as follows: (See Fig. 12).

Fig. 12



In Fig. 12, P is the earth pressure and W is the weight of earth and concrete as heretofore defined. Also:

$$ab = y \text{ Tan } \Theta, \text{ but Tan } \Theta = \frac{P}{W}, \text{ hence } ab = y\frac{P}{W}.$$

The resisting moment the wall offers against overturning about the toe point c, is then equal to the weight W times the arm

i.e. to $W\left(a + y\frac{P}{W}\right)$

Note: The value of the factor usually referred to as the overturning factor is equal to the resisting moment given above divided by the earth pressure moment Py. A study of the rate of variation of the value of the numerator and denominator of this fraction for various wall heights will show that for a given wall type the overturning factor will vary but slightly throughout the entire range of height.

Overturning design—Equating this resisting moment to the moment the wall is designed to withstand, established above, we have for:

Type 1 walls—$W\left(a + y\frac{P}{W}\right) = 1.75\ Py$

But for Type 1 walls $a = 0.355b$. (Fig. 6)

Making this substitution and solving for b,

$$b = 2.12\frac{Py}{W} \quad \text{.....(L)}$$

Type 2 walls—Similarly with $a = 0.25b$

$$b = 2.40\frac{Py}{W} \quad \text{.....(M)}$$

Type 3 walls—Similarly with $a = 0.429b$

$$b = 1.75\frac{Py}{W} \quad \text{.....(N)}$$

Type 4 walls—Similarly with $a = 0.500b$

$$b = 0.80 \frac{Py}{W} \quad \text{(O)}$$

Equations (L), (M), (N), and (O) will serve for determining the relation necessary between b and t to fulfill overturning requirements. Note, that all that is necessary to arrive at this relation, is to substitute in these equations for W its value in terms of b and t, previously established.

Now let us investigate overturning for the first example—the 33.50 ft. Type 2 wall on rock foundation for which values of $t = 4.70$ and $b = 14.0$ have already been determined to satisfy crushing and sliding requirements. The given data are as follows:

$W = 40900$ lb. $\qquad y \frac{P}{W} = 5.17$ ft.

$P = 17960$ ft. lb.

$Py = 211000$ ft. lb. $\qquad a = 0.25 \times 14.0 = 3.50$

Substitution in Eq. (M) gives $b = 12.4$ ft. So overturning does not govern the width of base required. The actual resisting moment of the wall against overturning is:

$$W\left(a + y\frac{P}{W}\right) = 40900\ (3.50 + 5.17) = 355{,}000 \text{ ft. lb.}$$

whereas only $1.60\ Py = 1.60 \times 211000 = 338{,}000$ ft. lb. is required.

Likewise for the 33.50 ft. Type 1 wall on earth foundation the wall will be found to offer 542,000 ft. lb. resistance whereas only 370,000 ft lb. are required. So here again overturning does not govern.

For the sake of solving a more general case let us suppose that the overturning requirement for the 33.5 ft. Type 2 wall on rock foundation had been $1.90\ Py$ so that the resistance moment of $1.68\ Py$ available with the 4.7 ft. toe and 14.00 ft. base would have been insufficient. The general overturning equation for such a case would be obtained as before by equating the moment of resistance about the toe to $1.90\ Py$ and solving for b.

$$b = 3.60 \frac{Py}{W}$$

An expression that shows the relation required between t and b to meet this overturning requirement may again be derived by substituting for W and Py in this equation the values previously established and determined; i.e.,

$$W = 3725b - 3200t + 3826, \text{ and } \quad Py = 17960 \times 11.76.$$

Making these substitutions in the equation $b = 3.60 \frac{Py}{W}$, and solving for b:

$$b = \sqrt{193 + (0.43t - 0.52)^2} + .43t - 0.52$$

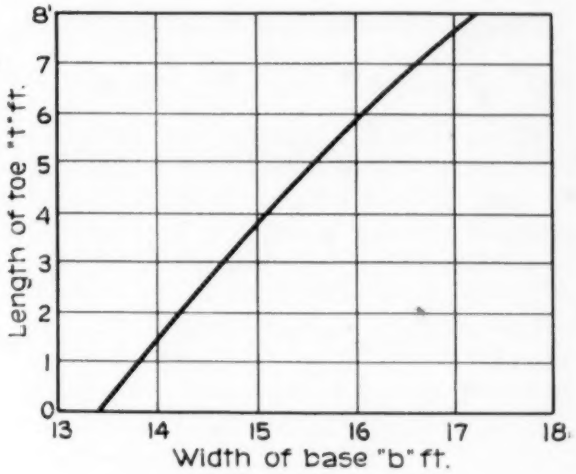


Fig. 13—Overturning graph

The graph of this equation is shown in Fig. 13.

Exactly to fulfill the overturning requirement then, the relation between b and t expressed in this equation must hold.

Solving this equation simultaneously or graphically with the crushing or settlement equation derived above, i.e.,

$$b = \sqrt{1.90t^2 - 4.56t + 221} - 0.43t + 0.52$$

gives the following results:

$t = 1.65$ ft. $\qquad b = 14.6$ ft.

The corresponding $W = 3725 \times 14.6 - 3200 \times 1.65 + 3825 = 53000$ lbs.

The ratio between W and $P = \dfrac{53000}{17960} = 2.95$; i.e., $W = 2.95\,P$, well over the $2.20\,P$ required for sliding.

These then are the values of t and b that would in that case fulfill all stability requirements.

Note that a simple graphical study could have been made to predetermine in the last case mentioned, that sliding could not govern. From such graphs the designer could have foretold at a glance that the simultaneous solution of the overturning and crushing equations would give the governing values.

Toe—The toe is a slab, cantilevered from the front face of the stem, loaded with the upward earth pressure minus its own dead weight. Since the earth on the heel may be placed before any fill is placed on the toe, no deduction is made for the fill on the toe.

The critical section for bending and punching shear is a vertical section at the front face of the stem.

The critical section for shear involving diagonal tension and for bond may be taken as a vertical section $\frac{2d}{3}$ in front of the front face of the stem.

Stirrups shall be avoided by increasing the toe thickness. The effect on stability of any such thickness change that may be necessary is negligible and will be on the safe side. Likewise any increase in footing thickness necessary to keep f_c within the allowable, has a negligible effect on stability and is on the side of safety.

The top of the footing may be sloped downward away from the stem in accordance with the rate of decrease of the governing requirement. The minimum thickness at the outer end should be from 16 in. to 2 ft. 0 in. depending upon estimated severity of construction impacts.

Heel—The heel is a slab, cantilevered from the rear face of the stem, loaded with the dead weight of the retained material directly above it plus its own dead weight.

For Type 1 and Type 2 walls, both of which have a triangular pressure diagram, the upward earth pressure shall be entirely disregarded. For these types the portion of the triangularly distributed upward earth pressure under the heel alone is comparatively small. Its neglect provides an additional margin of safety against earth pressure variations. For Types 3 and 4 walls an additional margin of safety equivalent to that for Types 1 and 2 may be obtained without disregarding the entire upward pressure on the heel. For these cases one-third and one-half respectively of the total upward pressure on the heel may be disregarded instead of the entire earth pressure and for sake of simplicity this reduction may be assumed uniformly distributed over the heel.

The critical section for all design requirements, i.e., bending, punching shear, and shear involving diagonal tension and bond, is a vertical section at the rear face of the stem.

Just as for the toe, stirrups shall be avoided by increasing the slab thickness. Also effect on stability may be disregarded if it is found necessary to change the footing thickness to meet the f_c value, u, etc., or for reasons of economy. Likewise, the top of the footing may be sloped downward away from the stem in accordance with the rate of decrease in the governing requirement. To withstand construction impact the minimum edge thickness should be 16 in. to 2 ft. 0. in., depending upon the estimated severity of construction impacts.

Additional safety precautions against sliding—Additional precautions against sliding may be had by:

(a) Sloping the bottom of the footing,

(b) Providing a concrete key between the bottom of the footing and the foundation soil.

(c) Omitting the toe form and pouring the footing directly against the soil.

Note that the value of expedients (a) and (b) above, is limited by the horizontal shearing value of the soil and that the effectiveness of expedient (c) is reduced by the initial shrinkage of the fresh concrete.

Footing reinforcement along bottom of heel—The designer should be on the alert for footing reinforcement requirements along the bottom of the heel—a requirement usually met by merely extending, say every other toe bar, towards the heel. This requirement is usually found necessary where the heel is comparatively long and thin, and for stresses calculated for loads occurring after the stem has been poured and the form braces removed but before any backfill has been placed.

Additional economies—The designer should be on the alert for the following additional economies:

(a) Use of higher strength concrete in the footing whenever the required footing thickness is greater than the established minimum thickness. Bond often governs the required thickness so that a higher strength concrete with its consequent higher allowable bond stress should in that case be investigated and a cost comparison made.

(b) Toe bars may be curved upward into the rear of the stem thus serving also as reinforcement for the lower section of the stem. This saving must be compared however, with the increased cost of bending, placing and supporting the reinforcement and with the fact that the amount of steel provided in the toe must be increased to satisfy the requirements at the base of the stem, as the stem requirement is usually appreciably greater than the toe requirement.

(c) For footings with comparatively short heels the heel thickness may be reduced and heel steel increased correspondingly. The toe thickness could then be stepped down under the stem and this step would provide an effective key between the footing and stem.

WALL TYPES, EARTH PRESSURE, CHARTS

Wall types

Having developed and adopted a design method, each office must then select the wall types for which charts should be prepared. Study of soil mechanics together with past experience may reveal that for a given region, Types 1 and 2 would suffice. This, for instance, would be true for many regions in Pennsylvania. For other localities, Types 1 and 3 might embrace the majority of walls, and so on, depending on the region and on how conservative the engineers' judgment may be. There will inevitably remain special cases such as those governed by relation between walls and property lines or by physical characteristics that would exclude them from any of the types referred to above.

In selecting these types, due consideration must be given to any appreciable decrease in the 35 ft. height limit. For instance, were the maximum height but 20 ft., the maximum pressure for Type 1 would decrease to two tons per sq. ft. It would evidently be unnecessary in such a case to resort to Type 3 walls.

Earth pressure

The determination of earth pressure values is beyond the scope of this paper. Far reaching results have recently been achieved in this field. For an outstandingly clear and concise treatise with special reference to retaining walls, the writer recommends Chapter X of Krynines "Soil Mechanics." Every designer should take time out to read at least this chapter. He will no doubt find it intensely interesting. However, insofar as this paper is concerned, it is immaterial what method is used in arriving at the magnitude and line of application of the earth pressure. A pressure based on the theory of Terzaghi, Spangler, Hogentogler, Boussinesque, Coulomb, Rankine, or any other theory which has proven sufficiently safe and economical may be used. The fact that the writer has based the few illustrated examples and charts given herein on Rankine's theory with two foot horizontal surcharge should not be construed to have any significance whatsoever. Significant, however, is the fact that in each of these cases it was definitely known that this manner of computing earth pressure had proven satsifactory for similar designs and in similar locations for the past 30 years or more. The fact that new developments tend to prove that the value of y, the arm of the pressure moment, should be increased to about $0.5H$, and that the value of P should also be slightly increased, indicates that the actual margins of safety were not as high as presumed. This, however, might be interpreted as a tribute to the engineering judgment used in the past in selecting safety margins and properly evaluating the limitations of the original Coulomb and Rankine theories. More precise knowledge acquired in future developments based on a close correlation between field observations and office design may make possible further reduction of safety margins and in turn greater economy of sections.

The following comparison is given purely as an interesting sidelight encountered by the writer in trying to determine why Rankine pressure values for a two ft. horizontal surcharge were often used for walls that were actually loaded, instead, with a sloping surcharge, (sloping to a definite height), such as is sometimes the case at the free end of wing walls.

The Coulomb pressures given below, Table 2, are for a 1½:1 sloping surcharge *indefinite* in extent, using an angle of 24 deg. 50 min. for the angle of friction between earth and concrete, and an angle of 42 deg. 30 min. for the angle of internal friction of earth.

TABLE 2

Overall Height Feet	Horizontal Component of Coulomb's Pressure Kips	Rankine's Pressure Kips	Comparison
15	4.3	4.3	Rankine's equals Coulomb's
20	7.7	6.9	Rankine's is 10% less
25	11.9	10.4	Rankine's is 13% less
30	16.9	14.6	Rankine's is 14% less
35	22.0	19.5	Rankine's is 11% less

The moments imposed on the wall about the base compare as follows:

Overall Height Feet	Coulomb's Moment Ft.-Kips	Rankine's Moment Ft.-Kips	Comparison
15	21.6	24.5	Rankine's 13% more
20	51.4	49.5	Rankine's 4% less
25	99.5	92.5	Rankine's 7% less
30	169.0	155.0	Rankine's 8% less
35	256.0	240.0	Rankine's 6% less

Charts

The final step is to compute the data and prepare the design charts.

Two charts, similar to Charts A-1 and A-2 below will suffice for the stem.

Chart A-1 serves to establish the batter of the rear face. Thus for a 23 ft. stem height the minimum rear face batter is seen to be $\frac{7}{8}$ in. per ft. Let us say the designer for that case establishes a batter of 1 in. per ft.

Chart A-2 serves for determining the amount of moment reinforcement. For the wall mentioned above with 1 in. per ft. rear face batter, the reinforcement required is seen to be 1.25 sq. in. per linear foot at the base of the stem, 0.90 sq. in. at an elevation 20 ft. below the top and so on for any other height.

The data needed for the tabulation given in Chart A-1 are obtained by substituting in Eq. (d) and (g), and the data for plotting the curves given in Chart A-2 are computed by either of the two methods explained and illustrated under the section entitled "Design of Any Section Above Base."

The equivalent fluid pressure surcharge mentioned in Chart A-2 is taken from the 1941 A. A. S. H. O. specifications which reads in part as follows:

Structures designed to retain fills shall be proportioned to withstand pressure as given by Rankine's formula; provided, however, that no structure shall be designed for less than an equivalent fluid pressure of 30 pounds per cubic foot.

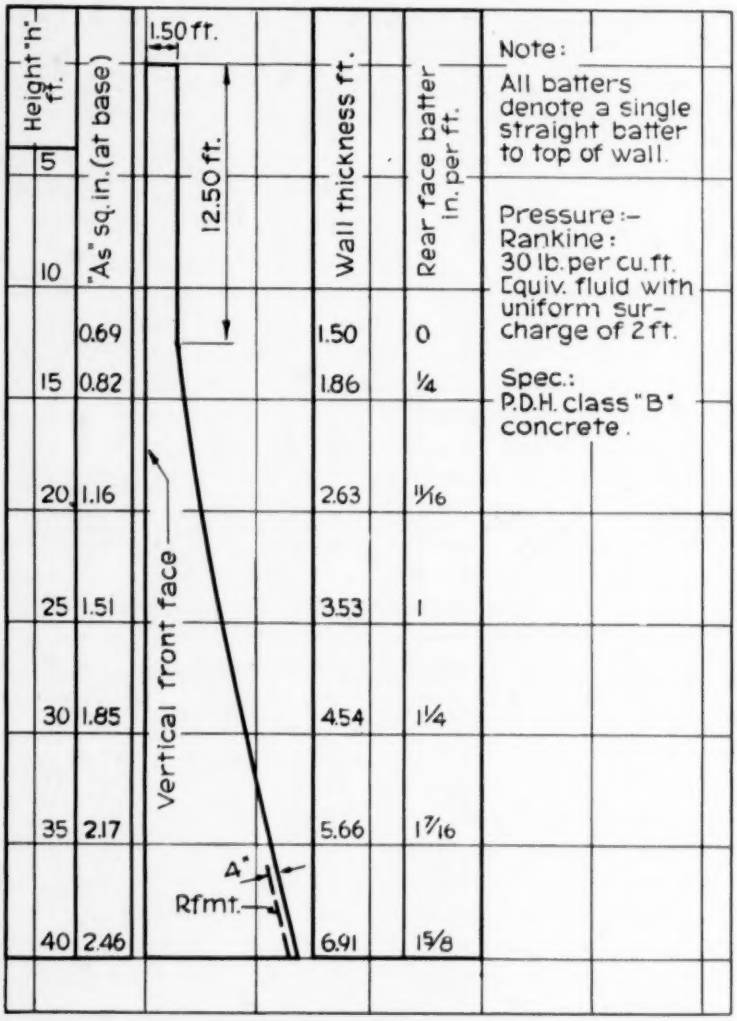


Chart A1

(Rear face batters $fc = 600$, $fs = 18000$, $n = 12$)

When highway traffic can come within a distance from the top of the structure equal to one-half its height, the pressure of 30 pounds per cubic foot shall have added to it a live load surcharge pressure equal not less than 2 feet of earth or the equivalent fluid pressure of 60 pounds per cubic foot.

Designers will recognize the fact that counterforts should be considered for walls above a 30 or 35 ft. height. However, the higher heights shown on the charts will be of value in special cases and for walls varying from a low to a 40 ft. height. Counterforted walls are not considered in this paper.

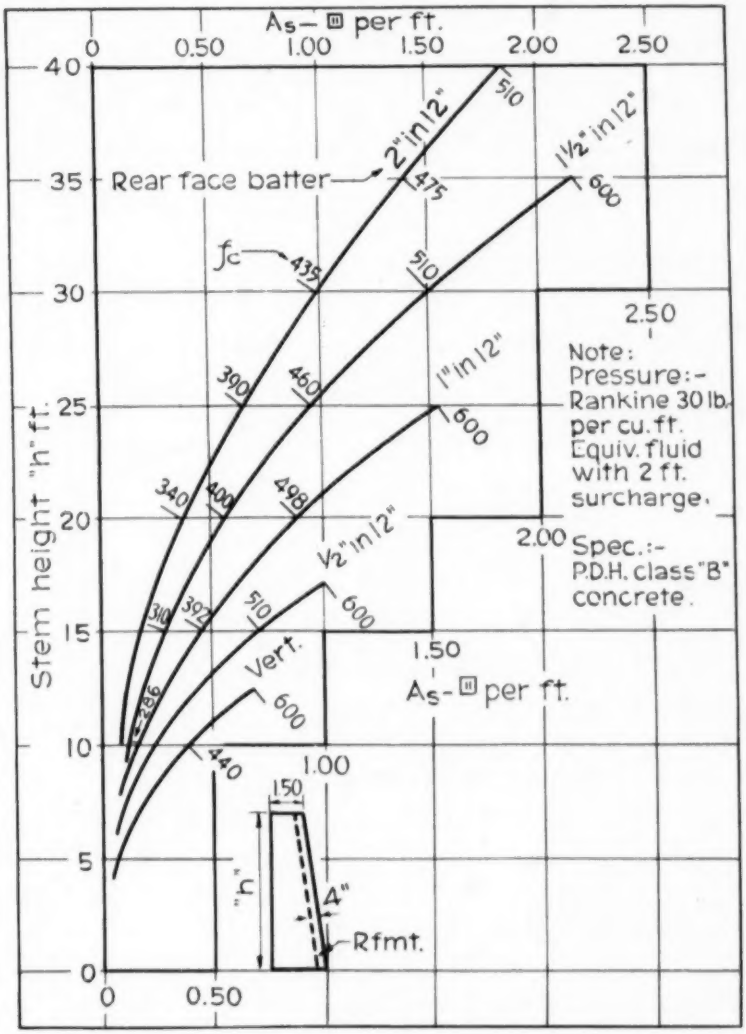


Chart A2

(Moment reinforcement, stem)

1 ft. 6 in. top, vertical front face, $fc = 600$, $fs = 18000$, $n = 12$

A close scrutiny of the rate of change of curvature of the graphs will help guide the designer in selecting the h values for which he should compute the plotting data. Note the number of values needed is *surprisingly few.*

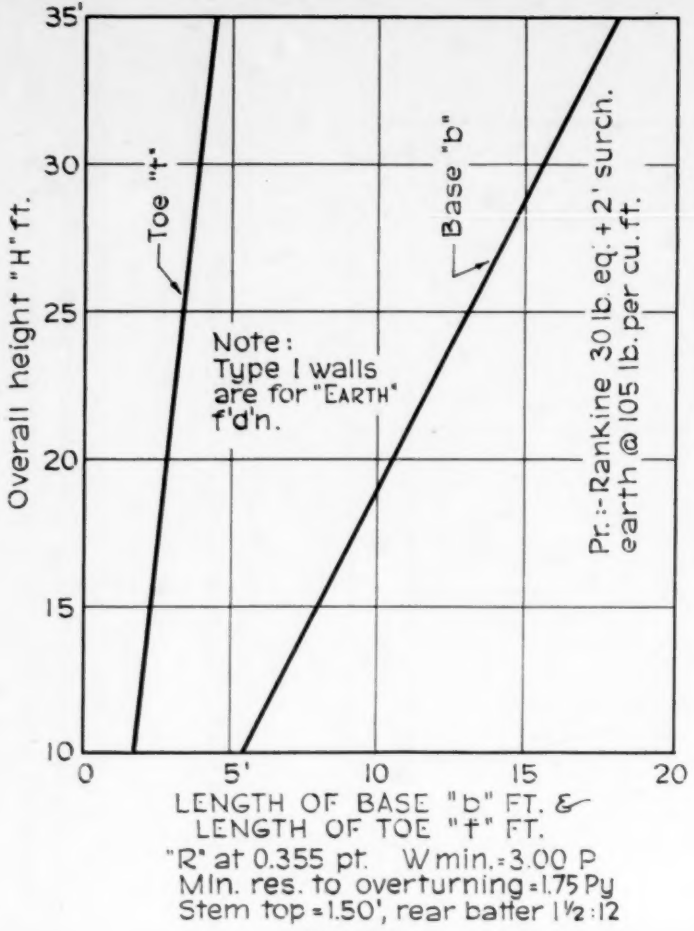


Chart B-O

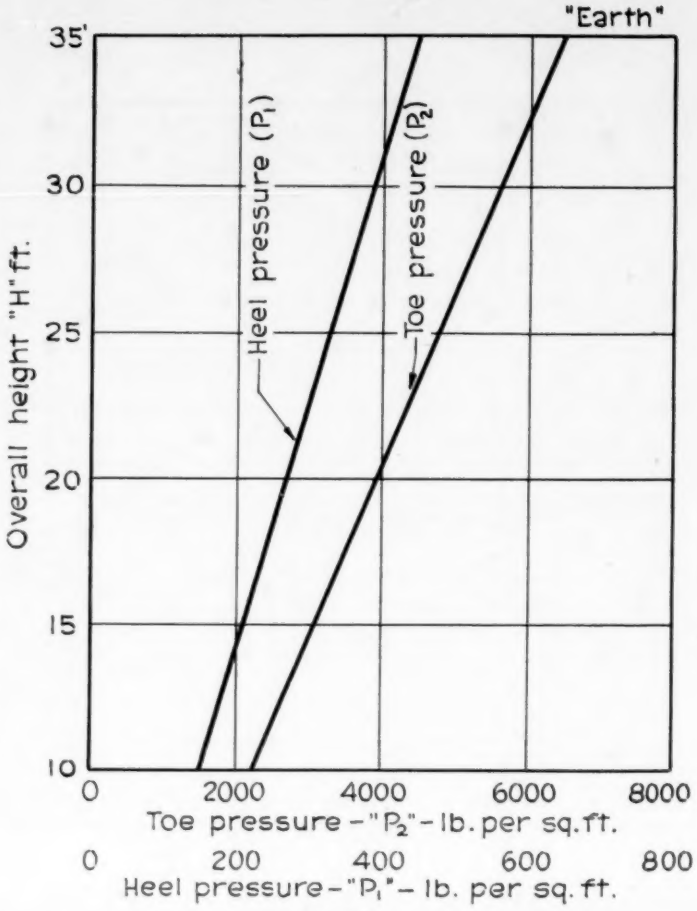


Chart C-O.

Chart B-O

Chart C-O.

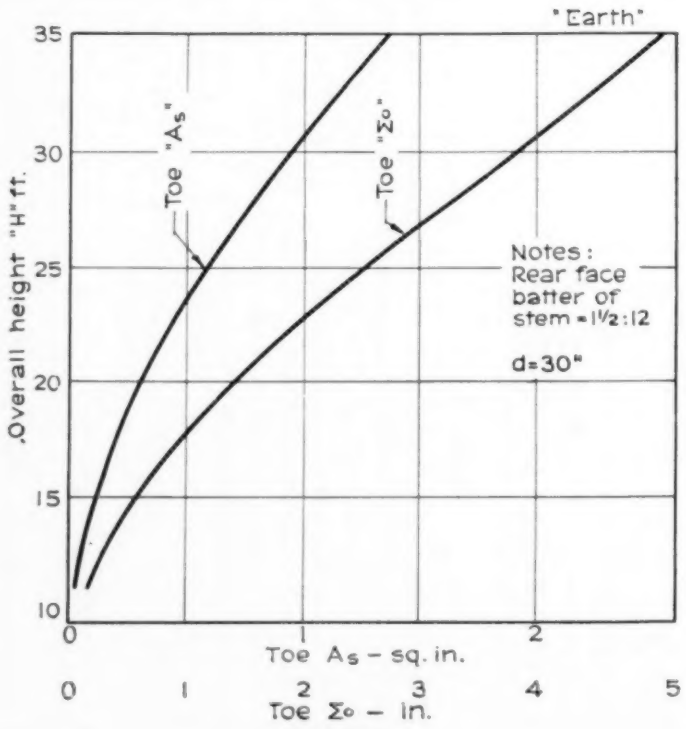


Chart D-O

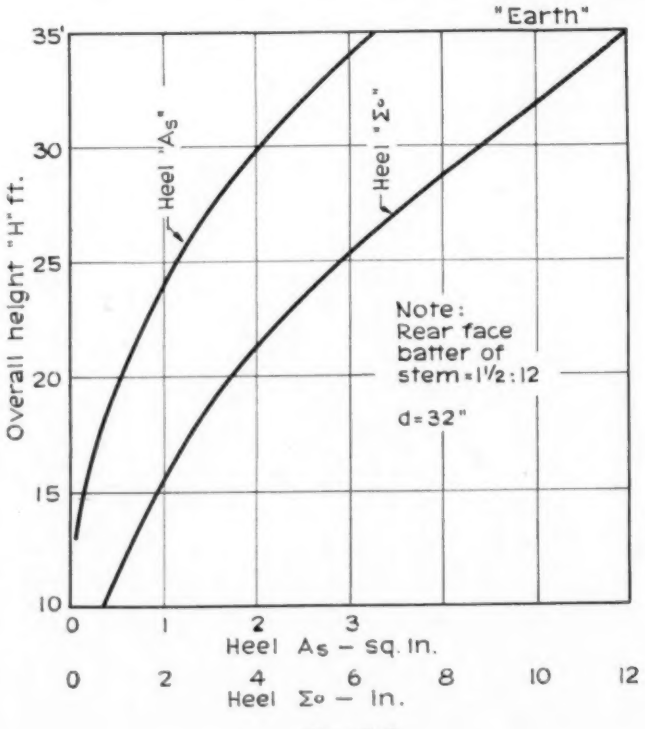


Chart E-O

Four charts, similar to Charts B-O, C-O, D-O, and E-O, will suffice for the footing.

Chart B-O—Chart B-O serves for determining the footing dimensions. The length of the toe and of the base are determined directly from this chart. The heel length is determined by first adding the toe length to the bottom thickness of the stem and then subtracting this sum from the length of the base.

Thus for a 26 ft. overall height wall of the type referred to on Chart B-O, the toe length would be 3 ft. 6 in. and the base 13 ft. 7 in. The toe length plus the bottom thickness of the stem would be 3 ft. 6 in. plus 4 ft. 4½ in. (4 ft. 4½ in. equals 18 in. top plus 1½ in. per ft. batter times 23 ft. stem height) equals 7 ft. 10½ in. The heel length then would be 13 ft. 7 in. minus 7 ft. 10½ in. or 5 ft. 8½ in.

Note also on Chart B-O the graphs are practically straight lines and hence *only two or three points are needed for plotting each graph.*

Chart C-O—This fact is also true of Chart C-O which serves for determining the toe and heel pressure. For the 26 ft. overall height wall referred to above, the toe pressure would be 4,900 lb. per sq. ft. and the heel pressure, 340 lb. per sq. ft.

Charts D-O and E-O—Charts D-O and E-O serve, respectively, for determining the amount of toe and heel reinforcement. On each chart the reinforcement required for both moment and bond is given and the governing requirement is readily determined therefrom.

Additional charts—Charts showing the sliding and overturning margin of safety actually provided, for any overall height H, can be easily prepared and added to those illustrated.

CLOSING WORD

The design method described herein will serve to point the way in design offices, to the development of a method of cantilever retaining wall design that will obviate the necessity of making preliminary assumptions of the dimensions of the structure. Such a method will eliminate office disputes concerning the ideal answer to each design problem; will encourage instead of discourage the preparation of design charts; and in turn will economize design labor.

The reader should understand that the factors and design charts herein are intended to serve primarily as illustrations for the design method, and are not to be used for other cases unless first proven conservatively suitable for the actual conditions.

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Supplementary Data on the Effect of Concrete Aggregate Having Low Thermal Coefficient of Expansion*

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Member American Concrete Institute

SYNOPSIS

A limited number of tests were made on 2 x 2 x 10-in. bars containing two types of low coefficient aggregates and one type of silica pebbles, which were submitted to cycles of temperature changes from —20 F. to room temperature. After about 50 cycles, bars containing the low coefficient aggregates lost 40 to 50 per cent of their original modulus of elasticity, whereas the bars containing silica pebbles lost only 2 or 3 per cent of their original modulus. Expansions during these cycles were not large but that of the low coefficient aggregate bars was 3 or 4 times that of the silica pebble bars. After 100 cycles of temperature changes, the bars were submitted to freezing and thawing cycles of the same temperature range as before. The bars containing the low coefficient aggregates succumbed very rapidly to the frost action while the silica bars were only moderately affected by the same treatment. The results lend further support to the conclusion of the earlier paper by the author, namely, that low thermal coefficient aggregates can be a source of danger in concrete exposed in cold climates.

An early failure of severely exposed cast stone, described by the writer in the September JOURNAL (1941), and discussed by a number of people in the June JOURNAL (1942), was attributed primarily to internal stresses and strains set up by the use of aggregate having an unusually low thermal coefficient of expansion and differing materially from the thermal coefficient of portland cement paste. Without precise knowledge of the thermal and mechanical characteristics of the matrix surrounding the aggregates, reasonable assumptions regarding these properties indicated the possibility of the development of high stresses at low temperatures and consequent cracking of the matrix, which, in the unfavorable environment, rendered the concrete extremely susceptible to frost action.

Some of the discussors of the paper were of the opinion that the severe exposure alone was sufficient to account for rapid failure, in spite of the

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fact that by commonly accepted standards this concrete was of exceptionally high quality, and should not have succumbed to frost action in a single winter's exposure. Some of the discussors, particularly F. V. Reagel and T. F. Willis, suggested that tests be made to determine the effects of low thermal coefficient apart from frost action. This notion was so intriguing that, in spite of conditions discouraging further immediate research on the subject, considerable thought was given to a simple program that would indicate whether concrete made with low thermal coefficient aggregate would suffer from temperature changes alone.

Unfortunately only small residues of the low coefficient aggregates remained from the original tests. Only enough Vycor glass (thermal coefficient 0.0000005 per degree in the range −20 to 180 F.) remained for making up two 2 x 2 x 10-in. bars, but four similar bars were made up from Maryland marble (thermal coefficient −0.0000012 in the range −25 to 75 F., +0.0000044 in the range 75 to 212 F.). For comparison four bars were also made with Ohio silica pebbles, (thermal coefficient of these particular pebbles was not determined, but that of similar material is known to be of the order of 0.000006 to 0.000007 per degree F.) The grading of these coarse aggregates was from No. 4 to ½-in., and the same matrix was used for all bars, consisting of fine Ottawa silica sand in a normal cement paste of 6 gal. per sack water-cement ratio. Enough of the various coarse aggregates was added to a fixed quantity of the matrix to give approximately the same consistency (2 to 3-in. slump.) Thus the test bars had the same water ratio and the same consistency, but the quantity of coarse aggregate varied in such manner that the cement contents per cubic yard of the three concretes were as follows: 5.64 sacks for the Vycor glass; 6.57 sacks for the Maryland marble; and 6.15 sacks for the Ohio pebbles. In general, a constant cement content as a basis for comparative durability tests is to be preferred, but in this particular case a fixed water ratio seemed desirable.

The test bars were cured 28 days in the damp room; then one of the marble bars and one of the pebble bars were immediately immersed in water, and freezing and thawing cycles started. It is important to note that these two bars did not at any time lose water by evaporation, were therefore as completely saturated as possible when the freezing started, and thereafter were always immersed. The remaining 8 bars (2 glass, 3 marble, 3 silica) were each wrapped with four layers of wrapping paper (ends exposed) and placed in a controlled room at 70 F. and 50 per cent R.H. to dry slowly. One layer of wrapping paper was removed at the end of each succeeding week and then the bars were left to dry until a very low weekly loss was observed. The point of all this was to minimize differential shrinkage stresses as much as possible by slow drying.

At the end of 8 weeks in the constant humidity room these 8 bars were weighed and measured and their sonic moduli of elasticity were determined. Then they were alternately placed dry in refrigerator air at −20 F. and in room air at 70 to 80 F., the schedule permitting two complete cycles in 24 hours. For the first 20 cycles the measurements were repeated after each 5 cycles, thereafter at the end of each 10 cycles. One minor effect of this treatment should have been foreseen and provided for, but it was overlooked. When the cold bars were taken from the refrigerator, a layer of frost began to form on the surfaces, and later this was melted and partly absorbed. This tended to dampen the surfaces of the bars and to increase their weight slightly; it may also have increased their lengths appreciably. As there were no suitable containers available for enclosing the specimens, this effect of frost formation was minimized by transferring the bars directly from the refrigerator to a ventilated drying oven maintained at about 110 F., and keeping them there about 45 minutes, or until the surfaces were slightly above room temperature, before returning them to the refrigerator.

In this manner the weight, length and sonic modulus changes were followed for 100 temperature cycles, after which the bars were immersed in water for 24 hours, and thereafter frozen and thawed immersed for an additional 40 cycles, or until failure occurred.

The most significant results were the length and sonic modulus changes, shown in the diagram. First, however, let us look at the curves marked M (wet) and S (wet). These refer to the single bars containing marble and silica aggregates that were submitted to freezing and thawing without loss of original mixing water. Note that the expansion of the marble bar was 0.2 per cent at 22 cycles, that of the silica bar 0.2 per cent at 44 cycles. The modulus of the former had been reduced to 50 per cent of its original value at 19 cycles, the same for the silica bar occurred at 25 cycles. It has been demonstrated numerous times in the writer's experience that when concrete is subjected to severe freezing and thawing with all its original water, its life is much shorter than when the same concrete has an opportunity to dry out before exposure to freezing. In this case the effect of thermal coefficient was largely overshadowed by the extreme severity of the test, and there was not a very marked difference between the two bars in resistance to disintegration. These tests were discontinued at 80 cycles, at which time the modulus of rupture of the marble aggregate bar was 58 psi, that of the silica aggregate bar was 101 psi.

Of more immediate interest in this limited investigation were the changes brought about by the temperature cycles from −20 F. to 75 F. It is seen that there was a moderate expansion of all bars, that of the marble and Vycor glass bars being above that of the silica bars, but not

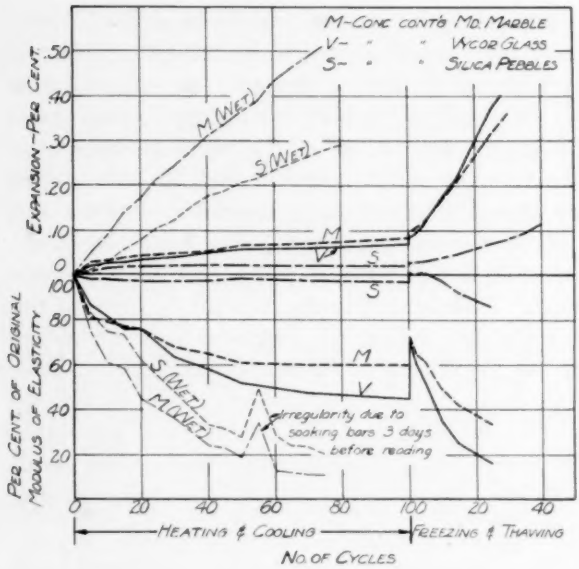

Fig. 1—Length and sonic modulus changes of concrete bars containing aggregates of different thermal coefficients of expansion after exposure to temperature and freezing and thawing cycles

To save space, the curves marked M (wet) and S (wet) are plotted in the portion of the diagram labeled "Heating and Cooling Cycles", whereas the two bars to which they apply were actually submitted to freezing and thawing cycles.

much above. Relatively a much greater difference was shown in the drop in sonic modulus of the three sets of bars, those containing the marble and Vycor glass dropping rapidly in the early cycles, while the silica bars showed very little drop from temperature changes alone. It is seen also that the marble and Vycor glass bars, which after 50 cycles had lost 40 to 50 per cent of their original sonic moduli, showed very little further loss up to 100 cycles.

A reasonable explanation of this behavior can be offered on the theory that was advanced for the original failure of cast stone. The concrete specimens were made, cured and dried at room temperature, the aggregates being held tightly in the hardened matrix. When exposed to the low temperature the matrix tended to contract, but was restrained and highly stressed by the low coefficient aggregates. This restraint may or may not have immediately produced cracks in the matrix, but with each additional cycle the chances of either cracking the matrix or of breaking the bond between matrix and aggregate were increased. Either of these developments would presumably account for the drop in sonic modulus, and that seems to be what happened to the marble and Vycor concretes, while the silica concrete remained unaffected because the aggregate has very nearly the same thermal movement as the matrix. One might well imagine also that the effect on overall expansion of the concrete from the differential movements of its ingredients would be quite small and only very slowly cumulative, which is seen to be the case.

At the completion of the 100 temperature cycles, the bars were soaked in water for 24 hours, and the weight, length and sonic modulus measurements repeated. It is seen that a very marked increase in modulus resulted temporarily from soaking the specimens, considerably greater than can be accounted for by taking into consideration the change in density. Then the freezing and thawing cycles were started, and the characteristic behavior of the concretes as noted in the earlier paper followed thereafter. It is seen that after 25 cycles of freezing and thawing, the sonic modulus of the marble and Vycor bars had fallen from 70 per cent of the original to about 33 and 16 per cent of the original, respectively, whereas the silica bars had dropped only to 86 per cent of their original value. Similarly, the expansion of the marble and Vycor bars had increased to nearly 0.4 per cent in 30 cycles of freezing and thawing, while the silica bars in the same time had expanded less than 0.1 per cent.

There was little change in weight of the dry concrete during the 100 temperature cycles. Actually there was observed a slight increase in weight (less than 1 per cent in all cases) which was attributed to the formation of frost on the cold bars, even when they were transferred to the oven at 110 F. Neglecting this small increase in weight, there was almost no loss until these bars had undergone at least 25 cycles of freezing and thawing. At this point the Vycor bars practically collapsed, and the same thing occurred somewhat less rapidly on the marble bars after 30 cycles. In contrast the silica bars showed only a few grams loss after 40 cycles. This behavior confirms the statement made in the earlier paper that concrete suffering from the type of malady under discussion shows almost negligible loss of weight until it is far along toward complete failure.

The results of these few tests have been reported in detail because they lend further support to the theory that aggregate of low thermal coefficient of expansion *can* be a source of danger in severely exposed concrete. Fortunately, most aggregates used in structural concrete are not in this category, but some limestones and marbles, frequently used in cast stone, are known to have low thermal coefficient at low temperatures. This type of aggregate should be under suspicion, particularly for use in exposed concrete in cold climates, until its thermal characteristics are known to be satisfactory.

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Reinforced Concrete Corners in Tension*

By C. J. POSEY† and ORVILLE KOFOID‡
Members American Concrete Institute

SYNOPSIS

Many concrete structures have corners which must resist bending moments that cause tension at the inside. Various ways of placing corner reinforcement for this loading condition are discussed herein, and the results of comparative tests are described which show that certain new designs are superior to currently standard designs in strength, toughness, and resistance to cracking.

Concrete corners subject to tension on the inside of the corner are found in structures of a wide variety of types. They are especially common in the field of hydraulic structures, with bench flumes, tanks, and square conduits furnishing familiar examples. In most cases the bending moment is greatest right at the corner, decreasing away from the corner in both directions. Design of the straight members which intersect to form the corner can be accomplished by methods which have been perfected during two-thirds of a century of experimentation and study. At the intersection, however, where the moment is greatest, the designer has no widely accepted standards to guide him, and but few experimental studies.**

DISCUSSION OF THE PROBLEM

An obvious method of reinforcing a corner for moment causing tension on the inside might be to determine, by the ordinary flexure theory, the amount of steel needed for the straight member immediately adjacent to the corner, and then put in bars providing this area, bending them as shown in Fig. 1(a). A corner so reinforced would be very weak. Fig. 1(b)

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**"Untersuchungen über die Bewehrung von winkelförmigen Eisenbetonkonstruktionen," G. Wästlund, *Beton und Eisen*, V. 35, n. 13, July 5, 1936, p. 222.
"Concrete Corners in Tension," D. B. Gumensky, *Engineering News-Record*, V. 123, Sept. 28, 1939, p. 411.

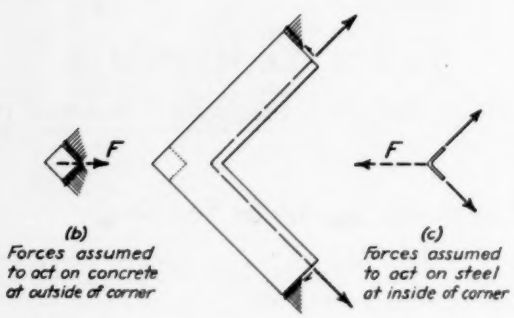


Fig. 1

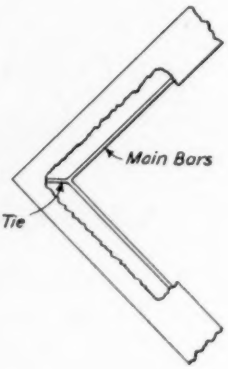


Fig. 2

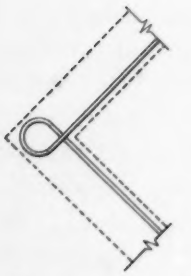


Fig. 3

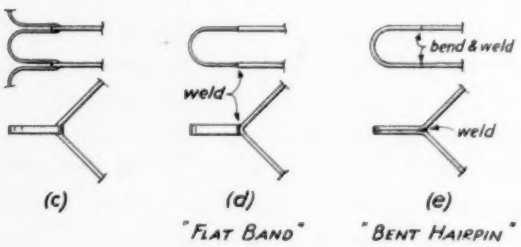


Fig. 4

Fig. 1-4

shows the forces tacitly assumed to act on the portion of the concrete whose position is shown by the dotted lines in Fig. 1(a). For equilibrium, a large force directed toward the right is needed. Since no steel is provided to take this force, the concrete at the outside of the corner will crack off. This effect is readily noticeable in experiments; even when steel is provided the concrete outside the steel cracks away.

Fig. 1(c) shows the forces tacitly assumed to act on the bars at the inside of the bend. A large force to the left is needed. As the concrete

is the only source of restraint, the encasement would break away before the stress in the main bars approached even a low working value.

The force needed by the concrete at the outside of the corner is numerically the same as that needed by the steel at the inside of the corner. Both can be taken care of by providing sufficient area of steel to tie the two areas together. Fig. 2 shows a sketch of a corner with such a tie, and with tensile areas of concrete omitted in the vicinity of the corner. If the strength of the corner is to be comparable with that of the straight intersecting members:

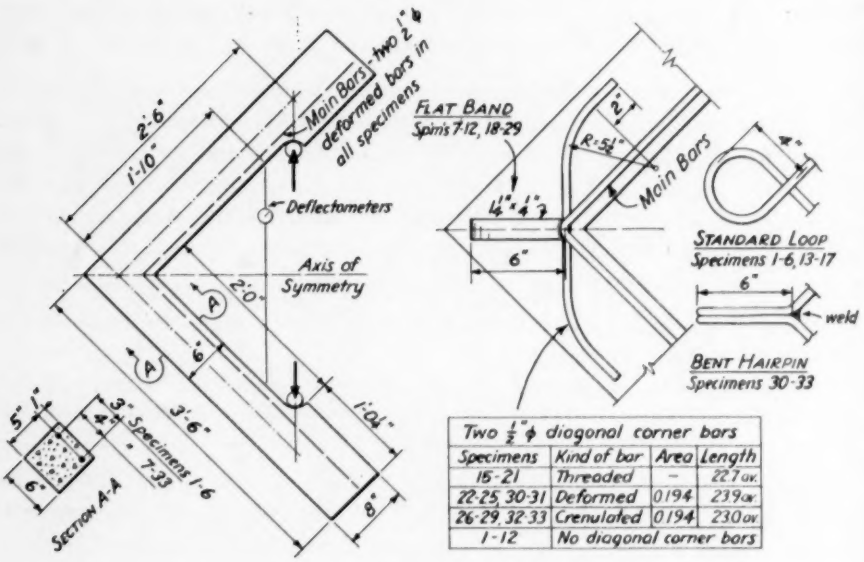
(1) the cross-sectional area of the tie must be at least $\sqrt{2}$ times the cross-sectional areas of the main bars;

(2) the tie must be fastened firmly at each end so that it can develop its full stress with a minimum of deformation and slippage. At one end it must engage the steel firmly, and at the other it must be securely anchored into the concrete.

Stirrups have a similar function, and it might seem that the tie can be made similar to an ordinary stirrup. While an ordinary stirrup will greatly improve the strength of the corner over that with no stirrup, it is not fully effective as a tie. The tie has to carry a far greater stress, in proportion to the main bars, than any beam stirrup, and it must do so in a very limited space.

The requirements outlined above are fairly well satisfied by the standard loop design shown in Fig. 3. The tie is provided by a looped extension of the main bars; thus the "connection" of the tie and main bars is free from slack or free play. Although the loops provide sufficient cross-sectional area for the tension in the tie, they are slightly less rigid than a straight tie would be. Anchorage in the concrete at the outside of the corner is accomplished in the way that would be most efficient, were it not for the fact that the loop is so oriented that it tends to split the concrete in a plane parallel to the direction of the main compressive stresses. Failure of concrete in compression often occurs by splitting along planes parallel to the stress. The two splitting tendencies reinforce each other.

If the loop is turned 90 deg. about a line bisecting the corner, its splitting tendency is directly opposed by the compressive stresses. It does not seem practical to accomplish this improvement by further bending of the main bars. If the standard loop is twisted 90 deg. to the position shown in Fig. 4(a), the bars are intertwined where they cross, with almost certain loss of rigidity through imperfect contact. By untwisting the standard loop 90 deg., and forcing the straight portions of the main bars into parallel planes one loop-diameter apart, the three-dimensional bend shown in Fig. 4(b) is formed. Its defect is that it requires two 45 deg. bends in the region of tensile stresses in the concrete.



Two 1/2" ϕ diagonal corner bars			
Specimens	Kind of bar	Area	Length
15-21	Threaded	–	22.7 av.
22-25, 30-31	Deformed	0.194	23.9 av.
26-29, 32-33	Crenulated	0.194	23.0 av.
1-12	No diagonal corner bars		

Fig. 5—Test specimens, with enlarged detail showing corner reinforcing

Another design embodying the principles stated, shown in Fig. 4(c), is a sort of continuous stirrup having the advantage of easy fabrication and erection of both bar and stirrup. Some means of securing tight contact with each main bar such as spot welding or a strong spring clip would be necessary with this scheme, however. The design shown at Fig. 4(d) is nearly the same as that shown at (c), but the U-stirrup is welded to a pair of main bars. Design (e) is made with ordinary round bars, first bent U-shape to a large radius, then bent sharply in a plane perpendicular to the plane of the first bend, and finally welded together in pairs. Designs (c), (d), and (e) have an advantage over the other designs in that the tie is shorter and more rigid.

Of the schemes for arranging the main reinforcement shown in Fig. 4, that shown at (d) was chosen for comparison with the standard loop design.

Designs 4(d), 4(e), and the standard loop were also compared in another arrangement with additional secondary bars placed diagonally across the corner as shown in Fig. 5. The purpose of the secondary bars was to help the resistance to cracking and increase the strength of the corners. It was considered important for them to develop a high stress with minimum slip. In practice, it is customary to hook these bars to achieve this. It has been shown, however, that the highest ratio of load to slip is obtained when the bar is bent as little as possible, and when

it has a surface that will develop high bond strength.* All of the diagonal bars were bent as shown in Fig. 5. Three different types of surface roughness were used.

Previous investigations have included designs with various sizes of fillets. A fillet will increase the cross-sectional area through the corner, and thus increase its strength and stiffness. If the line of action of the resultant tension is fixed, as for a statically determinate test specimen, the eccentricity to the corner cross-section will be smaller the larger the fillet. Filleting a corner in an indeterminate frame, however, will shift the line of action of the resultant tension away from the corner, due to the increased stiffness of the corner. Since in either case these effects can be evaluated with at least a fair degree of accuracy, the present investigation was restricted to comparing the various possible ways of placing the reinforcement for the un-filleted corner. For the related problem of concrete corners in compression, Richart and Olson concluded that "the strengthening effect of fillets was largely due to the reduction in bending moment and the slightly increased depth of member at the point of failure."†

It should be noted, in passing, that there is an essential difference in the theoretical shape for fillets for corners in tension and those in compression. The fillet for the corner in compression should be rounded if the best stress distribution is to be obtained, but the fillet in tension should be straight, for the steel bars running through it cannot follow a radius without provision for larger transverse forces than can be carried by the concrete. Filleting a sharp tension corner, then, changes it into two corners that are less sharp, so that the problem of reinforcing a filleted corner is similar to that of reinforcing a sharp corner, but less difficult.

FOUR SERIES OF TESTS

Four series of tests comparing the different designs were made in the Materials Testing Laboratory of the State University of Iowa, all under the supervision of the senior author. The necessary special apparatus was designed and built and the first series of 15 specimens (including numbers 1, 2, 7, and 8 of Table 1) tested by the junior author in the school year 1939-40. The second and third series, of 12 and 5 specimens (3-6, 9-12, 18-21, and 13-17 of Table 1) were tested by Daniel Sages in the spring and fall of 1941. The fourth series of 12 specimens (22-33 of Table 1) was tested in the spring of 1942 by Phillip Eisenberg, graduate

*"Tests of Anchorages for Reinforcing Bars," Chesley J. Posey, University of Iowa Studies in Engineering, Bulletin 3, 1933.

†Rapid and Long-Time Tests on Reinforced Concrete Knee Frames," by F. E. Richart and T. A. Olson, ACI JOURNAL, Mar-Apr. 1937, *Proceedings* V. 33, p. 478.

assistant in Civil Engineering. John Fielding and A. V. O'Brien assisted in the preparation of the apparatus and steel forms and Don R. Schamp and other N. Y. A. students assisted with the casting and testing. The project was part of the work of the College of Engineering and the Graduate College of the State University of Iowa. Dean F. M. Dawson is head of the College of Engineering, and Professors B. J. Lambert and J. W. Howe of the Departments of Civil Engineering and Mechanics and Hydraulics, respectively.

In order to be able to compare the strength properties of the different reinforcing schemes, the specimens were planned so that simultaneous values of total load and deformation could be obtained. In the first series of tests corners of two different sizes were tested, using two different percentages of reinforcement. The relative merits of the reinforcing schemes tried showed up similarly in both sizes, and with both percentages of steel. In the remainder of the tests only one size of specimen and one size of main bars were used, permitting the testing of a greater number of specimens. Except for the specimens that were the same size and had the same size of main reinforcing bars as the later series, the results of the first series of tests are not summarized here.*

Test details

In all, 34 specimens were cast having the same outside dimensions and same main reinforcing bars, but with several different arrangements of corner reinforcement. No data were secured for one specimen, which was accidentally broken. Data for the remainder are shown in Fig. 6 and Table 1. The specimens were cast on their sides, with both legs horizontal, in nearly watertight steel forms. The bars were held securely in place as shown in Fig. 8, permitting thorough tamping. Only specimens 7 and 8 showed signs of settlement away from the bars. They both had small cracks above and in line with the main bars which were only one inch below the surface. The molds were well oiled, but oil was not allowed to touch the bars.

The main bars were intermediate grade new billet steel, bent cold. The bars used in specimens 13, 14, 22, 23, 26 and 27, however, were not new, but were obtained by breaking up previously tested specimens. Since none of the steel was strained very far beyond the yield point, these used bars should have had very nearly the same properties as new bars. A comparison of the average results from specimens 22 and 23 or 26 and 27 (with used bars) and otherwise identical specimens 24 and 25 or 28 and 29 (with new bars) shows that this assumption is correct; the deviations are small and occur sometimes in one direction and sometimes in the other. All of the diagonal bars were intermediate grade new billet steel. The threaded bars were cut with an old bolt die, 13 threads to the inch. The crenulated bars were made by pounding the deformations into the cold bar with a square-pointed cold chisel. The flat bands welded to the main bars of specimens 7 to 12 and 18 to 29, after being bent hot, were of mild grade new billet steel. All welds were shielded electric arc, made by an experienced welder.

The aggregates were Iowa River sand, crushed limestone (3/4 in. maximum) and local brands of portland cement. All concrete was mixed in a small power mixer of about

*A complete report is available for loan from the Graduate College of the State University of Iowa, Iowa City. "Reinforcement of Concrete Flume Corners," by Orville Kofoid, thesis, State University of Iowa, June, 1940.

one cubic foot capacity. Aggregate stored in the laboratory became very dry, making adjustments for moisture content unnecessary, and the same mix by weight was used throughout. A batch was 16, 24, 64, and 76 pounds of water, cement, sand, and crushed rock, respectively, giving proportions by loose volume of 1:2.35:3.25 with a water cement ratio of 1.00. Two control cylinders were cast with each specimen, and cylinders and specimens were cured in the laboratory by wrapping with burlap and sprinkling nearly every day.

All specimens except numbers 12 and 17 were tested 28 days after pouring, and these two were tested at 29 and 27 days, respectively. The control cylinders were tested on the same day as the specimens they accompanied. For the test, specimens were raised into position with a chain hoist and hooked into specially made hangers that carried the load to the crossheads of the 200,000-lb. capacity universal 3-screw lever type testing machine. A 20,000-lb. counter poise was used in the machine which was operated at a speed of 0.05 or 0.10 inches per minute. The hangers turned freely on pins at each end so that no restraining moment was introduced into the legs of the specimens. This freedom of movement at the ends permitted the specimen to rotate slightly due to its own weight. The actual eccentricity of the resultant force on the corner cross-section was therefore not fixed, but varied according to the load. The magnitude of this force was equal to the measured load plus the weight of the lower hanger plus half the weight of the specimen, or $P + 36 + 130$ lb. In plotting the results of the tes ts the measured load P was used, without any correction for eccentricity or dead load.

Before starting the test, the corner area was given a coat of plaster of paris paint. Three persons were required for a test, one to run the machine and read the load, one to read the deflectometer and record results, and one to watch for cracks and mark their progress. Simultaneous readings of deflection and load were taken, first at equal increments of load, and later at equal increments of deflection.

TEST RESULTS

Results of the tests are shown in Fig. 6 and Table 1. Although several different brands of cement were used, and corners of the same type were cast by different persons, reasonably comparable results were obtained, as is evidenced by the constancy of control cylinder strengths and the small amount of variation in observed values for identical specimens. The average ultimate strength of all control cylinders was 3500 psi. The lowest average for any group accompanying identical specimens was 3040, and the highest, 3900 psi. These variations are small, considering the natural variability of control cylinder strengths, and the small number of cylinders averaged. Correcting the strength values for the corner specimens in accordance with the average value of their control cylinders does not alter the comparisons appreciably; therefore it has not been done.

Fig. 6 provides a graphic comparison of the value of the different designs of corner reinforcement. To assist in comparing the merits of minor variations in the reinforcing, the area under each curve up to a deflection of one-half inch has been planimetered, and is shown in the column of Table 1 headed "work to deflect ½ in." Where the test was stopped before this deflection was reached, the curve was extrapolated

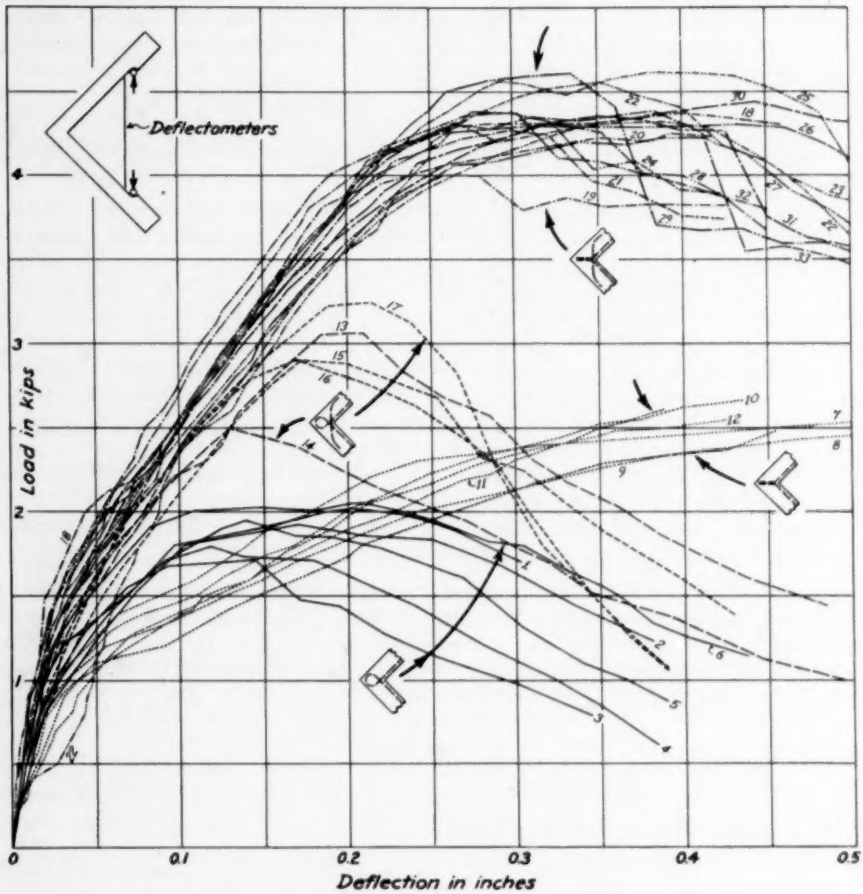


Fig. 6—Load deflection curves

to obtain a comparable value. This area is a measure of the toughness of the corner. The trend of the curves shows that the comparisons listed would be further accentuated if the deflection limit were to be extended past one-half inch.

Except for the values of load at the formation of the first crack, no crack records are shown here. The first crack invariably formed at the sharp reëntrant corner. It usually progressed inward slowly, branching out and following the lines of the bars. The concrete at the outside edge of the corner usually cracked away as the higher loads were reached. The specimens reinforced with standard loops showed cracks on the back of the specimen, in the plane of the loop. After these cracks formed,

TABLE 1

Arrangement of Reinforcing Bars	Specimen number	Control Cylinders No.	Control Cylinders Av. Ult. lbs/in²	Load at First Crack lbs	Ultimate Load lbs	Work to deflect ½ inch in-lbs
	1	4	3375	1000	2010	760
	2	4	3400	1000	1920	720
	3	2	3400	800	1800	510
	4	2	3300	1100	1720	550
	5	2	3800	1100	1950	650
	6	2	4400	1100	2010	810
	7	4	3420	900	2700	1020
	8	4	3440	800	2620	910
	9	2	3800	700	2500	930
	10	2	3700	800	2800	1030
	11	2	3800	1100	2800	990
	12	2	3500	1100	2600	990
deformed	13	2	3955	1100	3060	1050
	14	2	3435	900	2500	860
threaded	15	2	4080	700	2900	990
	16	2	3930	1400	2900	900
	17	1	3700	900	3240	940
threaded	18	2	3900	1600	4300	1780
	19	2	3400	1200	4000	1590
	20	2	4300	1400	4240	1700
	21	2	3600	1300	4270	1650
deformed	22	2	3240	1400	4550	1670
	23	2	3090	1000	4320	1690
	24	2	3135	1600	4380	1720
	25	2	3315	1000	4610	1810
crenulated	26	2	3255	1600	4300	1760
	27	2	3210	900	4220	1670
	28	2	3170	1000	4380	1690
	29	2	3190	1100	4610	1710
deformed	30	2	3190	1400	4440	1740
	31	2	2890	1000	4350	1700
crenulated	32	2	3550	1700	4310	1740
	33	2	3145	1200	4380	1730

the specimens failed rapidly, with decreasing load and increasing deformation.

In the group of specimens with flat bands or double bars reinforced with diagonal bars, cracks would start into the straight members from one to four inches away from the sharp inside corners. These cracks would first appear at about twice the load at which the corner crack appeared, but would then open faster and become larger than the corner crack. This seemed to indicate that except for the one factor of crackless strength, the reinforcing scheme of specimens 18 to 33 was very nearly

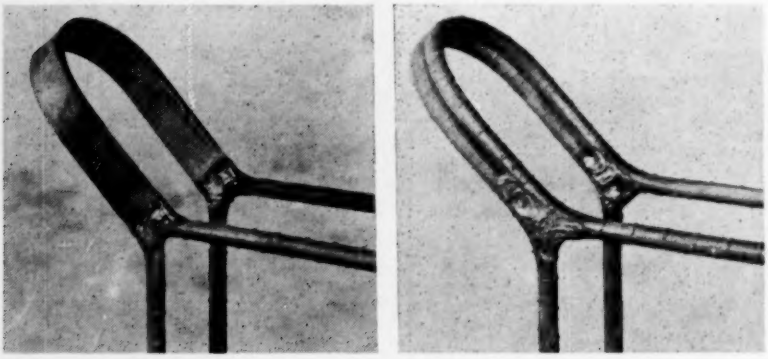


Fig. 7—Best types of main reinforcing tested; (a) flat band, at left, (b) bent hairpin, at right

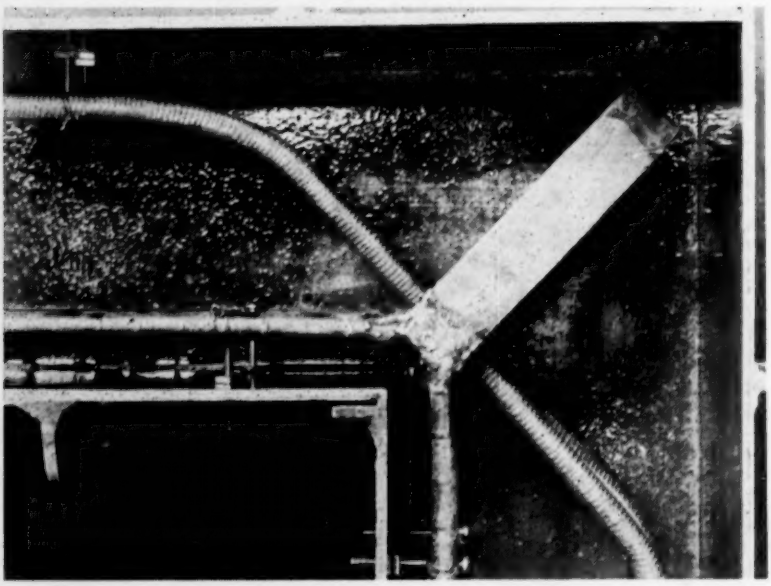


Fig. 8—Flat band with crenulated diagonal bars in forms ready for pouring

ideal, that is, the corner was as strong, or slightly stronger, than the straight members on each side.

A study of Fig. 6 and Table 1 indicates that the following conclusions are justified:

1. The flat band and bent hairpin designs with diagonal bars are superior to all others in crackless strength, in ultimate strength, and in toughness.

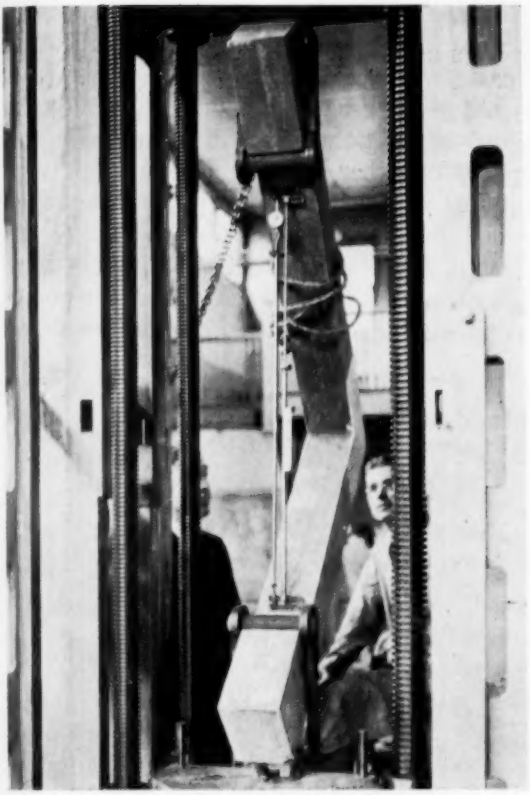


Fig. 9—Large specimen of the first series in machine ready for test

2. The standard loop design without diagonal bars has the lowest ultimate load and toughness. It is, however, superior to the flat band without diagonal bars in crackless strength and initial stiffness. The writers have not found a satisfactory explanation for this inferiority of the flat band without diagonal bars. It may be because the flat band, when cast in a horizontal position, traps bubbles in the wet concrete (dissection of tested specimens showed such bubbles, but no noticeable crushing of the presumably weakened concrete) or it may be because the cross-section of the flat band used was too wide and thin.

3. The standard loop design with diagonal bars is inferior in all respects to the flat band and bent hairpin designs with diagonal bars. It is superior to the designs without diagonal bars except in toughness. The load passes a maximum at a small deflection, and then falls off rapidly.

4. The various diagonal corner bars used have about the same value in improving the strength of the corners. It was intended to make all diagonal corner bars the same length. Bars removed from the dissected

specimens showed variations from the desired value of as much as 1.87 inch. The measured lengths are shown in Fig. 5. None of the bars showed any sign of having passed the yield point. The cross-sectional area of the threaded bars is not given because a short distance in the middle of each bar (1½-2½ in.) was not threaded. Their net effective area was probably less than that of either the crenulated or deformed bars. The small differences in the lengths of the bars do not seem enough to explain the failure of the high-bond surfaces to show definite superiority. Another possible reason is that the diagonal bars may not have been as carefully placed as the main bars. Fig. 8 shows that the crenulated bars were not quite in position, giving a smaller effective depth than they should have had.

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Chart for the Design and Adjustment of Concrete Mixes*

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Member American Concrete Institute

SYNOPSIS

The chart presented herewith permits the direct expression of the three variables of a concrete mix. These variables, cement, aggregate, and water are expressed in amounts per cubic yard of concrete, the expressions being given in those terms made familiar through long usage by the man in the field. The chart lends itself readily to the solution of the problems encountered in the adjustment of a mix.

The absolute volume relationship existing between concrete ingredients and their weights and specific gravities, now generally utilized in the study of concrete mixtures, has been the basis for numerous time-saving and error-minimizing devises, probably the foremost of which is the nomographic chart.‡ The usual form in which this chart is presented when dealing with concrete mixes will be found to employ a terminology which many men in the field do not normally use in their problem solutions. For example, the water-cement ratio and water content are expressed in terms of weight in place of the more common and usual job practice of considering the water in terms of gallons per sack of cement and gallons per cubic yard of concrete. Also, the amount of aggregate will not be found expressed in terms per cubic yard of concrete. The terms usually encountered are, "Parts of aggregate to cement by weight" or "Absolute volume per sack of cement." Such form of expression does not permit direct reading of the aggregate quantity, demanding certain arithmetical steps before a figure is obtained in the units to be employed in the batching of the aggregate. The chart herewith presented obviates the necessity for such conversion of terms and its simplicity of construction enables the user to visualize graphically the changes incident to any concrete mix as arbitrary values are assigned to any of the variables.

*Received by the Institute Jan. 28, 1943
†Concrete Inspector, Erie Railroad Co., Jersey City, N. J.
‡For an example, see Discussion by Arthur Ruettgers of Report of Committee 613: "Proposed Recommended Practice for the Design of Concrete Mixes," ACI JOURNAL, June, 1942, *Proceedings* V. 38, p. 208-16.

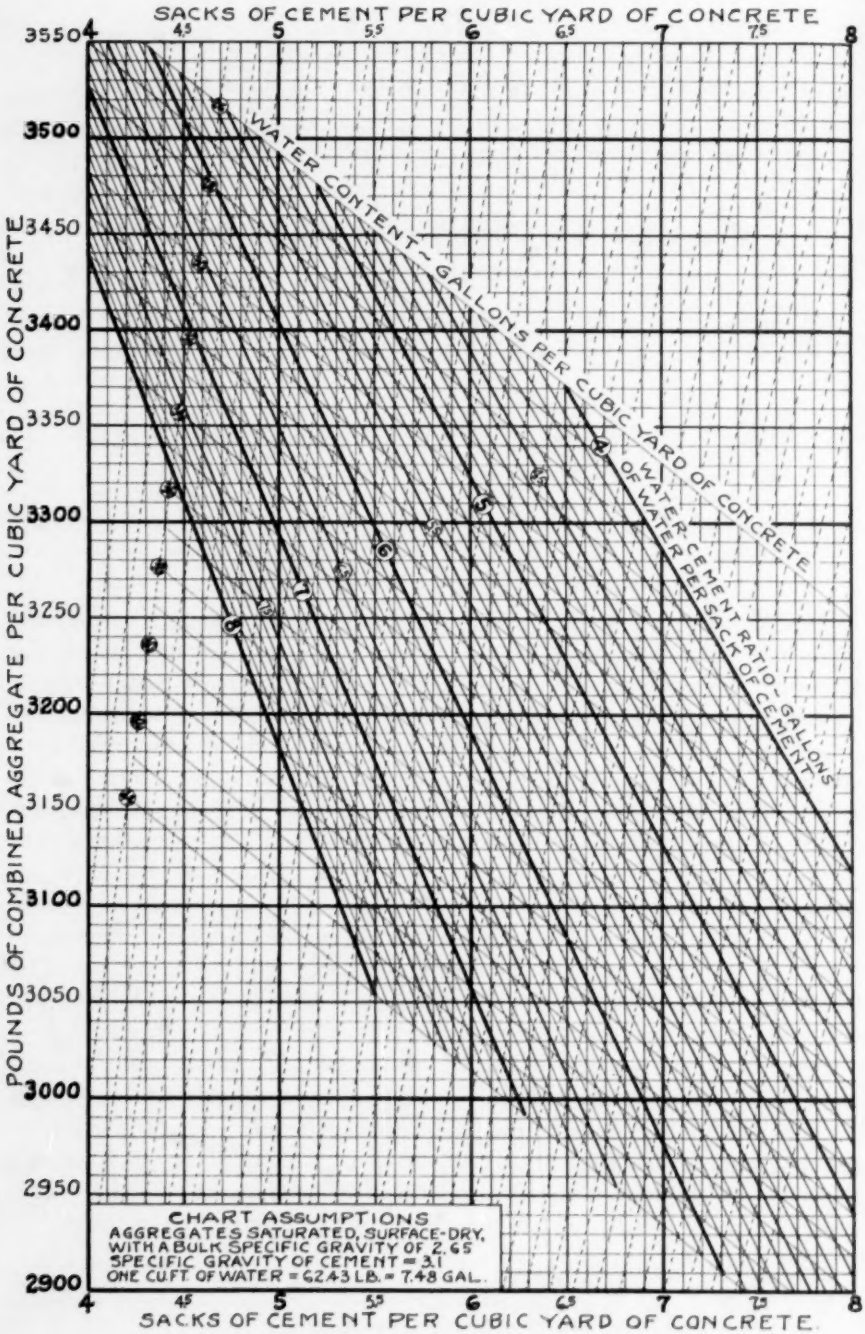

Fig. 1—Materials for one cubic yard of concrete

Chart construction

In concrete that contains no foaming agent, the amount of entrained air may be ignored for all practical purposes and by considering our concrete ingredients in terms of one cubic yard of mixed concrete we can derive the formula,

$$W = 4467 - 22.1\,C\left(\frac{w}{c} + 3.63\right)$$

Where W = Weight in pounds of combined fine and coarse aggregate in a saturated, surface-dry condition, the bulk specific gravity of which is assumed at 2.65.

C = The number of 94 pound sacks of cement, the specific gravity of which is assumed at 3.1.

$\frac{w}{c}$ = The water-cement ratio expressed in U. S. gallons of water per sack of cement.

Referring to the chart, it will be observed that the graph of "W" has been plotted for water-cement ratios in the range between four gallons per sack and eight gallons per sack in one-tenth gallon increments. The quantity "C" has been laid off as abscissae, the scale divisions of which permit reading direct to one-tenth sack of cement. The quantity "W" forms the ordinate system, the smallest scale division representing ten pounds of aggregate. The graphs have been confined to that area of the chart bounded by such water and cement contents that could normally be expected to result in the extremes of workable mixtures. For all practical mixtures the user will find himself well within the limits of the chart system. Every point upon the chart within the range of the water-cement ratios shown represents one cubic yard of mixed concrete. Thus, with any two of the three variables—cement, aggregate, or water-cement ratio—known, the third variable can be read direct.

The dotted lines extending from lower left to upper right have their origin at 0 sacks of cement and 0 pounds of aggregate and form a system of guide lines that permits moving from one part of the chart to another without disturbing the relative proportions of cement and aggregate in any particular mix under consideration. To illustrate, starting at the intersection of 6 sacks of cement and 3000 lb. of aggregate (500 lb. per sack), move along the diagonal guide line passing through this point. It will be seen that this guide line passes through the points of intersection representing 6.5 sacks, 3250 lb. and 7 sacks, 3500 lb. This relationship of 500 lb. of aggregate per sack of cement is maintained throughout every point on the line. Similarly, each dotted guide line—or any imaginary line visually interpolated between those offered for guidance—represents a definite weight of aggregate per sack of cement. Now note that as one

moves upward to the right along any guide line, the graphs of various water-cement ratios are intersected in turn, the upward movement having the effect of reducing the water-cement ratio of the concrete made up of that particular proportion of cement and aggregate represented by the guide line along which movement is being made. Thus, if we are given the number of sacks of cement and weight of aggregate going into any batch of concrete, (the volume of the batch may be less, equal to, or more than one cubic yard), by moving along the guide line from the point of intersection representing this information, we can read the quantities of materials required for one cubic yard of such concrete for any desired water-cement ratio.

The diagonal lines labelled, "Water Content—Gallons per Cubic Yard of Concrete," have been plotted in increments of 1 gal. for the range between 26 and 44 gal. per cu. yd. of concrete. Since water content is equal to the product of cement content and water-cement ratio, the location of the line of any water content is determined by the points of cement content and water cement ratio that are common to the water content under consideration. The lines of this system, each of which denotes a constant water content, can also be considered as lines of constant slump since the consistency of concrete mixes remains practically constant over a wide range of water-cement ratios if the type and gradation of the aggregates and the water content per unit volume of concrete remain constant. Now referring to the chart and remembering that every point thereon represents one cubic yard of concrete, as we move along any line of constant water content we can note the changes occurring in the cement and aggregate content of the mix caused by our passage through the range of water-cement ratios. Such movement across the face of the chart has the effect of defining a considerable number of one cubic yard mixes, each with a different cement and aggregate content as well as a different water-cement ratio, but each cubic yard so defined having in common with its group a constant slump. This feature can be used to advantage when it is desired to adjust the water-cement ratio of any known mix without disturbing the slump. Likewise, if the water content corresponding to any desired slump is known, a rapid study can be made of the combinations possible with such water content.

It must be remembered that the chart is based on the assumption that the specific gravities of the aggregates are equal to 2.65. To convert any weight of aggregate obtained from the chart to that of a different specific gravity, multiply the chart weight by the ratio, $\frac{\text{s.g. of aggregate.}}{2.65}$ Before entering the chart for purposes of study of any given mix containing an

aggregate of specific gravity other than 2.65, multiply the given weight of aggregate by the ratio, $\frac{2.65}{\text{s.g. of aggregate}}$.

Typical examples showing use of the chart

In the examples that follow, it will be assumed that the fine aggregate is to constitute 40 per cent of the total aggregate by absolute volume; that the bulk specific gravities of the saturated, surface-dry fine and coarse aggregates are 2.60 and 2.80 respectively;

Example 1—Given: W/C ratio of 6 gal. per sack.

Cement content to equal 6 sacks per cu. yd. of concrete.
Water content equals 6 gal. x 6 sacks = 36 gal.

Required: Saturated, surface-dry weights of fine and coarse aggregate per cu. yd. of concrete.

At the intersection of the vertical line representing 6 sacks per cu. yd. with the line of water-cement ratio labeled 6 gal. per sack of cement, read the weight of aggregate, 3190 lb.

Fine aggregate = 40 per cent of 3190 lb = 1276 lb.
Coarse aggregate = 3190 − 1276 = 1914 lb.

Correcting the weights due to the specific gravities being other than the chart assumption of 2.65;

$$\text{Fine aggregate} = \frac{2.60}{2.65} \times 1276 = 1252 \text{ lb.}$$

$$\text{Coarse aggregate} = \frac{2.80}{2.65} \times 1914 = 2022 \text{ lb.}$$

Example 2—Assume that a stiffer consistency is desired after noting the appearance of the concrete produced with the amounts of ingredients developed in example 1. This stiffer consistency can be obtained by reducing the amount of water being added at the mixer. This is done and we find that the desired consistency requires a total water content of 30 gal. for the batch. The questions now demanding answer are as follows:

(a) What quantities of materials are going into each cu. yd. of concrete with our 6 sack batch using 30 gal. of water in place of the original 36 gal.?

(b) What volume of concrete is our 6 sack batch producing with 30 gal. of water?

Our water-cement ratio has been reduced from 6 gal. per sack to $\frac{30 \text{ gal.}}{6 \text{ sacks}} = 5$ gal. per sack. From the point of intersection of 6 sacks, 3190 lb., (the chart equivalent of the 3274 lb. of aggregate going into the batch, example 1), proceed radially upward toward the right, adjacent to the dotted guide line, until the water-cement ratio line of 5 gal. per scak is intersected. Read,

(a) 6.18 sacks: 3288 lb. combined aggregate.
Water content = 6.18 × 5 = 30.9 gal. water.

(b) Volume of the 6 sack batch equals $\frac{6}{6.18} = 0.97$ cu. yd.

Example 3—It now becomes desirable to know what changes must be made in the mix quantities in order to get back to the original water-cement ratio of 6 gal. per sack and still maintain the consistency obtained in example 2. Proceed as follows:

From the intersection 6.18 cement, 3288 lb. aggregate, move parallel to the water content lines along a visually interpolated line of water content equal to 30.9 gal.

Where this line intersects the W/C ratio line of 6 gal. per sack read,

5.15 sacks cement: 3370 lb. aggregate.

Similarly, the changes required for any other W/C ratio or any desired cement content could have been read as we moved along the line of constant water content which in this case was equal to 30.9 gallons. The aggregate weight of 3370 lb. must be corrected for specific gravity as in Example 1.

Example 4—A mixer is being charged with 5 sacks of cement and amounts of aggregate which, when reduced to saturated, surface-dry weights, equal 1177 lb. fine aggregate, and 1900 lb. coarse aggregate. What is the volume of concrete being produced if a water-cement ratio of 7 gal. per sack is being maintained?

Converting the weights of aggregate into terms of a specific gravity of 2.65,

$$\frac{2.65}{2.60} \times 1177 \text{ lb.} = 1199 \text{ lb.}$$

$$\frac{2.65}{2.80} \times 1900 \text{ lb.} = 1798 \text{ lb.}$$

Combined aggregate = 2997 lb., say 3000 lb.

Enter the chart at the intersection of 5 sacks, 3000 lb; proceed along the dotted guide line to its intersection with the W/C ratio line of 7 gal. per sack; read 5.35 sacks per cu. yd.

Then the volume of concrete produced by the 5 sack batch equals,

$$\frac{5}{5.35} = 0.93 \text{ cu. yd.}$$

In conclusion, the effect of movements across the face of the chart upon the ingredients of any one cubic yard mix under consideration may be summarized as follows:

1—Vertical movement along a line of cement content holds the cement content constant and trades water for aggregate in maintaining the volume of concrete at one cubic yard. The water-cement ratio and slump during such vertical movement are under continuous change.

2—Horizontal movement along a line of aggregate weight holds the aggregate content constant and trades water for cement in maintaining the volume of concrete at one cubic yard. The water-cement ratio and slump are under continuous change with such movement.

3—Movement along any line of water-cement ratio holds the W/C ratio of the paste constant and trades the cement paste for aggregate in maintaining the volume of concrete at one cubic yard. The slump is under continuous change during such movement.

4—Radial movement in the system of dotted lines holds the proportion of cement to aggregate constant and trades such combined cement and aggregate for water in maintaining the volume of concrete at one cubic yard. The W/C ratio and slump are under continuous change during such movement.

5—Diagonal movement, parallel or coincident with any line of water content, holds the water content constant and trades cement for aggregate in maintaining the volume of concrete at one cubic yard. Such movement causes a continuous change in the W/C ratio but leaves the slump unchanged.

Discussion of this paper should reach the ACI Secretary in triplicate by March 1, 1944 for publication in the JOURNAL for June 1944.

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Job Problems and Practice

"JPP" is a means toward realizing more fully than in a limited number of longer JOURNAL contributions, the mutuality of ACI Membership effort to do a better concrete job. In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved—in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

The Use of Coral Aggregate (40-138)

Q—It is not surprising that there have been many inquiries about the use of coral as concrete aggregate. A typical inquiry was referred to Mr. Nutter who writes from a store of experience.

By BEN E. NUTTER*

A—"Can we use coral as concrete aggregate?" The answer, at so many locations, is, "You have no alternative if you are to have concrete." That might conclude the story.

The more encouraging answer, however, is an enthusiastic "yes." Coral is now being used as concrete aggregate for everything from A to Z; small building footings that require but little strength; mass concrete, protecting vital elements; precast concrete piling; revetments; protection bunkers; culverts; crew shelters and pavements. The concrete which results, however, is as variable as the thought and planning and work which go into it.

This subject should be divided into three sections. First, there are those who will be making concrete in the active war zones. The con-

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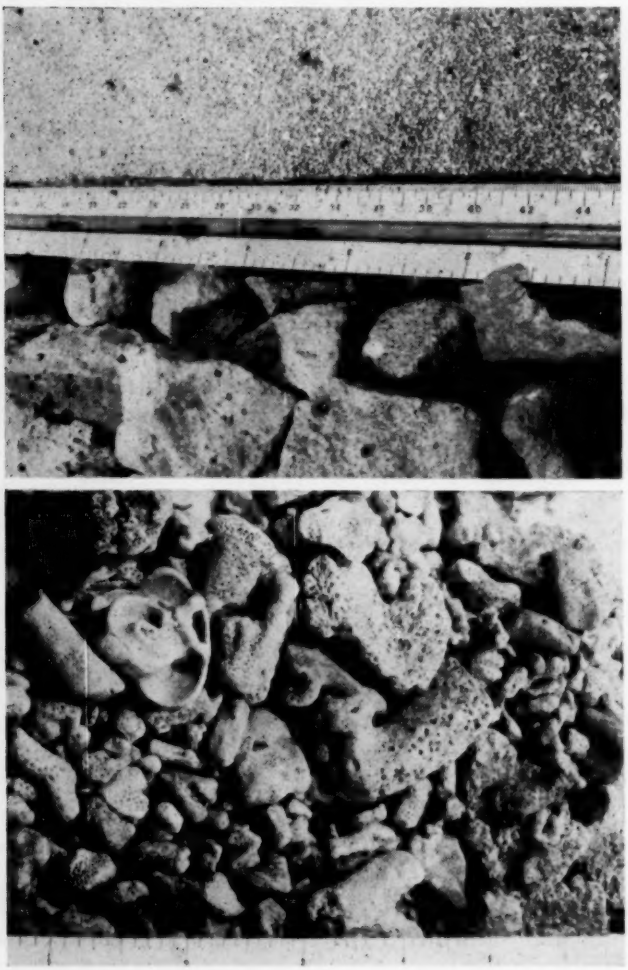

Fig. 1—Coral aggregates

struction here will be touch and go of whatever bank run material is handy. Second, there will be other places where the moderate amount of concrete required will justify a selection between materials, and a measure of care in construction. Third, at some major stations it will be found economical of time and materials to have a plant which can at least screen the coral, or perhaps wash or crush it, and to take the care required to establish a considerable measure of control.

Coral occurs in all manner of sizes and gradings. The same principles apply to the final grading of aggregate for coral concrete as do for conventional concrete and therefore selection or processing is desirable. There is normally an opportunity for selection even on the smallest of atolls.

The side of the island normally leeward will offer one size of material while the windward may offer another. The beach dunes may offer a needed size or the bottom of a lagoon just offshore may be used to advantage if a clamshell or dragline is available. A bay or inlet may offer a source of material rather uniform in size. A little prospecting frequently pays good dividends.

It is well to select materials containing but little material passing the 100 and 200 mesh sieves if the coral is at all soft, or if crushed coral is used. Many of the softer types grind in the concrete mixer to produce ample quantities of these sizes in the final mix.

The uses of coral for concrete have been many. A hard, dense coral can make satisfactory precast concrete piling. The concrete should be designed for a strength of 3500 to 4000 psi. and considerable density. A minimum water content and ample water curing followed by air curing for carbonation of the surface will insure maximum durability. For protective structures the highest quality of available aggregate will provide the greatest protection for a minimum expenditure of cement and reinforcing steel. In using coral concrete for pavements the hardest available varieties should be used and the concrete finished in such a manner that maximum abrasion resistance will be obtained; curing is again an important factor. Foundations and floors for small temporary buildings, where strength is not critical, can make use of the lower grades of coral if such use is required. For any use, the best aggregate available, will permit the greatest economy of cement.

Coral hardness and toughness can be conveniently judged by its specific gravity. Most organizations charged with any form of construction will have equipment which will permit one with ingenuity to determine the specific gravity of the material under consideration. The material of the highest specific gravity should be selected, as that will produce the highest grade of concrete. There is a large variation in specific gravity and hardness of coral, even between adjacent locations. Aggregates of a specific gravity greater than 2.20 will be found excellent in strength producing characteristics if they are clean and uncontaminated but those below 2.00 can not be depended upon to produce high strength concrete. This does not imply that 2.0 is the lower limit for the specific gravity of usable coral.

Confirmation of the selection of a source may usually be obtained by sending samples of the materials back, for abrasion tests, to the nearest service laboratory, Army or Navy. Air transport planes frequently return lightly loaded, permitting surprisingly quick results to be obtained from a well equipped laboratory. Test cylinders from the job or from preliminary trial mixtures may also be sent to these central agencies and their advice obtained concerning specific problems that arise.

Many of the special characteristics of coral as aggregates will be of interest only to those using it with a large measure of control and in relatively important structures. The absorption is high. It may exceed ten per cent for the fine aggregate and five or six per cent for the coarse sizes. The measurement of absorption of fine aggregate by obtaining the saturated surface dry condition through the use of the sand cone has not been found very satisfactory. The method of using a hydrometer to measure the dilution of a standard solution of calcium chloride by the free water or surface water on the sample is believed to give a truer picture of the amount of surface moisture and, indirectly, the absorption. Fine coral frequently has two characteristics which cause inaccuracies in the cone method. The coral is porous and blotter-like and does not reach a saturated surface dry condition. At the time the aggregate is surface dry, the interior is no longer saturated. There is frequently enough cohesion to cause the cone of sand to stand even after the surface dry condition is reached. The coarse aggregate saturated surface dry condition is easily obtained by the hand drying method if it is done with care and done quickly.

The writer has not found evidence of excessive expansion or contraction attributable to the use of coral, although excessive use of water will cause the same effects in coral concrete as in concrete made with conventional materials.

The impurities to be found are principally guano, vegetable matter and salt. The organic matter must be avoided. Sea water in the aggregate does not cause serious loss of strength at the early ages and no evidence has yet been found to show serious deterioration caused by it at later ages. In fact there are a few old concrete structures exposed to sea water in these outlying places that are remarkably well preserved. Legend has it that some of these were mixed with sea water; and the story is probably true.

One of the predominant characteristics of crushed coral is its tendency to grind in the mixer and to produce material finer than 100 mesh. If the amount becomes too great the mixture will require excessive water. The resultant cracking from shrinkage will resemble that condition in conventional concrete.

Dense, well-graded mixtures of coral aggregate will make concrete of strengths in excess of 4000 psi if it is required and if ample cement is easily available. The softer types of coral will make 2000 lb. concrete with relative ease but coral as an aggregate is not fool-proof. Strengths as low as 800 psi at 28 days (when 3000 was intended) have been recorded where normal selection, care and control was neglected.

In the batching of coral fine aggregate by volume, bulking is an important factor. Unit weights at each moisture condition must be obtained, in

order that the bulking may be known and the volume adjusted. Current changes in batch quantity for bulking are as important as moisture compensation.

The sizes of aggregate frequently found convenient are those corresponding to a sand size, usually processed through a ¼-in. vibrating screen, and a single size of coarse aggregate processed over a ¼-in. vibrating and through a 1¼-in. or 1½-in. vibrating screen of square openings. Coral at some locations lends itself well to the production of aggregate in a third size up to 2¼ or 2½ in., and this has been done, but the larger sizes cause low strengths when the soft grades are used. Internal vibrator units of a size appropriate to the job are invaluable in using the stiffer consistencies which will enable the consumption of cement to be minimized.

It is, perhaps, a little foreign to the subject to speak of cement in an article on coral, however, it is important that cement, carried perhaps as much as halfway around the world, be of a type as efficient as it is possible to obtain. The emergency alternate specifications for cement have permitted, if not required, a lower grade of cement. To combat this, procurement agencies must use the greatest care in selecting those brands of cement from manufacturers which elect to use the upper rather than the lower limits of quality and those brands which resemble the efficient A.S.T.M. Type II cement or Federal SS-C-206a, for all of the uses normally requiring standard portland cement. I believe it is not necessary to elaborate on the fact that cement for such shipments should be packed in the toughest, tightest, driest containers obtainable.

Bond in Isolated Footings (40-139)

Q—A question that has vexed designers in reinforced concrete is the treatment of bond in isolated footings. The question is whether bond must be treated as a function of shear or may be regarded merely as a means for anchoring a bar to prevent its pulling out of the projected part of the footing when stressed to capacity. Insistence on the first view may require more than double the quantity of steel reinforcing needed to satisfy the second, and except in very soft soils would mean that the concession of 15 per cent from the statical computation of bending made in both ACI and Joint Committee specifications was an empty gesture. For the bars could then rarely be stressed to capacity in bending.

For example, a half-inch deformed bar with a standard hook, in a footing of 2000-lb. concrete, designed for bond as a function of shear according to ACI Code (318-41) and Joint Committee specifications, cannot be stressed to capacity in bending unless the footing projects 4 ft.-5 in. from the face of the column. That is, the projection must be at least 105

bar diameters. But if shear may safely be ignored provided the bar with its hook will not pull out, the projection need be only 22 in. In this I assume the hook develops 10,000 psi and the remaining 10,000 is developed by 17 in. of embedment at 75 psi (25 per cent off 100 psi because of tension in two directions in the concrete). I have added 3 in. for protection and 2 in. for radius of the hook, making a total projection of 22 in.

Is there data in recorded tests of footings, or elsewhere, which would indicate that design of these bars thus for anchorage regardless of shear is not safe and proper?

By T. D. MYLREA*

A—Your correspondent has touched upon an extremely important application of the bond formula—one which may have to be faced again soon.

To begin with, practically all the experimental evidence as to the behavior of footings under load is that contained in Bulletin 67 of the University of Illinois Engineering Experiment Station, by the late Dr. A. N. Talbot, published in 1913, and it is upon this evidence that code requirements are largely based.

In Bulletin 67 are reported the results of tests made upon 114 wall footings, 93 of which were reinforced, and 83 isolated column footings, 75 of which were reinforced. Since the question concerns only isolated footings, the tests of wall footings will not be discussed. Of the 75 isolated footings, 59 were reinforced with plain round bars, and of these 31 failed by bond. The remaining 16 were reinforced with corregated bars, and of these only one failed by bond. This failure occurred at a very high computed bond stress, even though the bars were not hooked. Concerning these bond failures Bulletin 67 has to say:

> An important conclusion of the tests is that bond resistance is one of the most important features of strength of column footings, and probably much more important than has been appreciated by the average designer. The calculations of bond stress in footings of ordinary dimensions where large reinforcing bars are used show that the bond stress may be the governing element of strength. The tests show that in multiple-way reinforcement a special phenomenon affects the problem and that lower bond resistance may be found in footings than in beams. Longitudinal cracks form under and along the reinforcing bar due to the stretch in the reinforcing bars which extend in another direction, and these cracks act to reduce the bond resistance. The development of these cracks along the reinforcing bars must be expected in service under high tensile stresses, and low working bond stresses should be selected. An advantage will be found in placing under the bars a thickness of concrete of two inches, or better three inches, for footings of the size ordinarily used in buildings.

In the face of this evidence codemakers would necessarily be cautious. It is true that deformed bars behaved well in the tests and that deformed

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bars are the rule today. But the corrugated bars of 1908 to 1912 were followed by many inferior types of deformed bars. It is true, too, that our codes now require the three inches of concrete beneath the bars suggested in Bulletin 67, but the further tests suggested by implication in the closing paragraph of the bulletin have never been undertaken. Codes must be based upon direct evidence or upon unassailable reasoning.

On the other hand, there is much to say in favor of your correspondent's contention. In failing, a footing breaks up into a number of blocks. Fine cracks so spaced that the tension in the steel varies directly with the applied moment do not form. It may be reasonable to suppose, then, that pull-out phenomena would prevail. In this event, the average bond over a given length would be the deciding factor, and the average nominal bond on a uniformly loaded cantilever is only one-half as great as the maximum. This reasoning was followed "by some" when Bulletin 67 was written, and has since been used with apparent safety by competent engineers; and the latter fact constitutes evidence.

But in using a method based entirely on logic the designer should be sure that his logic fits such facts as are obtainable. The method suggested is shown in Fig. 1(a). Here, if a crack should form on the line CD the moment about point D in a unit width of footing would be

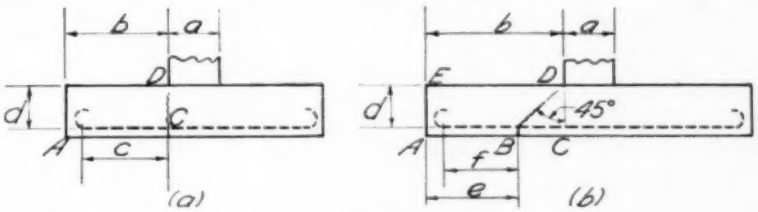

Fig. 1

$$M = p.b.\frac{b}{2}$$

and the tension in the steel at C would need to be developed by bond in the length c plus whatever might be assigned to the hooks. But a crack might just as easily begin at B, Fig. 1(b), running upward at an angle of about 45 deg. The failures described in Bulletin 67 lead to the conclusion that this is the normal type of failure. If so, the moment of the pressure under the block ABDE would create a moment *about point D* of:

$$M' = p.e.\left(b - \frac{e}{2}\right)$$

and the tension in the steel *at point B* would be

$$T = \frac{M'}{jd}$$

This tension would have to be developed by bond in the short length of plus whatever could be developed by the hooks. In the second example the saving ascribable to the pull-out method might easily fall below expectations, particularly if heavy soil pressures were used.

Incidentally the 15 per cent concession in the required steel area is not an empty gesture, since (ACI Code, Sec. 1205e) the bond unit stress may be computed from 85 per cent of the external shear.

The profession is in need of test data which will settle the question of whether, and under what soil pressures, and for what footing sizes, the pull-out method controls footing design.

"Influence of Sands, Cements and Manipulation upon the Resistance of Concrete to Freezing and Thawing" (40-140)

By F. V. REAGEL*

In the June, 1943 issue of the ACI JOURNAL there are discussions by A. D. Conrow and W. C. Hansen of the latter's paper, "Influence of Sands, Cements and Manipulation Upon the Resistance of Concrete to Freezing and Thawing."† In these discussions, the investigations of the Missouri State Highway Department on freezing and thawing tests of concrete are referred to by both Mr. Conrow and Mr. Hansen to support directly contradictory conclusions. Paradoxical as it may seem, probably both of these gentlemen are right in their conclusions. Discussion of this paper having closed it is too late to attempt to reconcile their conclusions. One point however, I would like to clear up. In the second paragraph on page 124-7, Mr. Hansen writes:

> F. V. Reagel (Proceedings the Highway Research Board 20-587, 1940), in discussing *loss in weight* and linear expansions, states "It may be found from further investigation that either or both of these measurements may serve as a check on strength measurements or even as an alternate method of determining deterioration."

The statement quoted from my paper did not refer to *loss in weight* as Mr. Hansen states but to *change in weight.* Actually for the tests described in my paper there was only one set of specimens which showed a *loss in weight* and these specimens showed the smallest expansion and had the greatest resistance to freezing and thawing. In the tests I described the relation between change in weight, expansion, and resistance to freezing and thawing was as follows: the greater the *increase* in weight the greater the expansion and the less the resistance to freezing and thawing. This is exactly the opposite of the relation found by Mr. Hansen. However, I am certain that if all the conditions, surrounding and influencing the two sets of tests, were evaluated, the apparent contradiction would disappear.

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†ACI JOURNAL, Nov. 1942; *Proceedings* V. 39, p. 105.

I hope that some way can be found to correct the impression which Mr. Hansen's quotation of my paper would undoubtedly leave with a reader who was unfamiliar with the original paper.

How to Determine Relative Workability (40-141)

Q—Can anyone supply a simple method for the determination of the relative workability of concrete mixes? "Workability" we define as the ability of newly mixed concrete to deform under externally applied force. Someone has no doubt arrived at a satisfactory method. The slump test does not meet the case because (a) there is virtually zero slump in many concrete mixes as used in concrete products practice; (b) the slump is not a true measure of workability; (c) at best it is a rough test of consistency only, subject to human error in tamping and in lifting off the cone; (d) the slump does not measure accurately the relative ability of various concretes to work into the required shape by forces such as tamping or vibration.

By R. F. BLANKS*

A—I am in agreement with the correspondent's comments (a), (b), (c), and (d). However, observations as to rodability, finishing qualities, and segregation (including bleeding) made during and immediately following the filling of the slump cone are very helpful in judging workability. These characteristics may be classified as "poor," "fair," "good," "very good", and "excellent." I suggest reading T. C. Powers' "Studies of Workability of Concrete," ACI JOURNAL, Feb. 1932; *Proceedings* V. 28, p. 419, discussion on page 693; also his paper, "The Bleeding of Portland Cement Paste, Mortar, and Concrete," ACI JOURNAL, June 1939; *Proceedings* V. 35, p. 465 (discussion on page 480-1).

By J. C. PEARSON†

This matter of a simple method for determining relative workability of concrete mixtures is an important one, and I have tried many times to find a satisfactory answer, without much success. The general scheme of attack has been to start with concrete of a given slump, and then do something else to the mix that is in some way designed to bring out its inherent plasticity. It is surprising how difficult this is to do, and we have not succeeded in getting much more than qualitative indications. I do not find that a remolding test of the Powers type will do this, nor the simpler scheme suggested by Goldbeck. In the case of masonry mortars a water retention test has been a pretty good index of workability, and the various bleeding tests of concrete have been promis-

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†Director of Research, Lehigh Portland Cement Co., Allentown, Pa.

ing. However, it would be more satisfactory to find a mechanical test of some sort, as the water retention test is not a direct measure of the property, and will never be accepted as entirely satisfactory until a mechanical test demonstrates its possible limitations.

By T. C. POWERS*

There are several devices of the restricted-flow type† which seem to be giving reasonably satisfactory results. Those which employ a vibrator have been used successfully on mixes having virtually no slump. Among these may be mentioned the following: The remolding test as modified by Bahrner (*Zement*, v. 29, p. 102-106, 1940); the vibrator-driven remolding test as used by C. E. Wuerpel at the Central Concrete Laboratory of the U. S. Engineer Corps, Mount Vernon, New York (unpublished);‡ the device developed by Hutchinson (A.S.T.M. Bulletin No. 99, p. 17, August, 1939).

For plastic mixes, we regard the slump test as a suitable measure of "the ability of newly mixed concrete to deform under externally applied force." It is sensitive to variations in water content, but not to variations in aggregate gradation. Consistent results can be obtained when proper attention is given to details of manipulation.

Vibratory Remolding Test as a Measure of Concrete Workability (40-142)

By CHARLES E. WUERPEL**

The term "workability" as applied to concrete mixtures has defied precise definition, but in its most practical application the term is used to express the relative facility with which concrete deposited in the form may be consolidated into its final position. This "relative facility" is called "relative remoldability" in this laboratory and is measured by the "remolding" apparatus.

The measurement of workability by means of a remolding apparatus was presented initially, and in considerable detail by T. C. Powers. The fundamentals of the Powers apparatus and procedure have been retained by this laboratory, but the remolding effort is accomplished here by external vibration in lieu of a flow table and it is measured in terms of time (in seconds) of vibration in lieu of the number of drops of a flow table.

The remolding apparatus assembly shown in Fig. 1 consists of a cylindrical metal container 12 in. in diameter and 8 in. high in which is sus-

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†See "Workability of Concrete" in Report on Significance of Tests of Concrete and Concrete Aggregates, American Society for Testing Materials, 1943.
‡See below—"The Vibratory Remolding Test as a Measure of Concrete Workability."—EDITOR.
**Principal Engineer, Central Concrete Laboratory, North Atlantic Division, Corps of Engineers, U. S. Army, Mt. Vernon, N. Y.

pended an inner ring having an inside diameter of 8¼ in. and a depth of 5 in. The inner ring may be adjusted so that the clearance between the bottom of the container and the lower edge of the ring may be varied from 2½ in. to 3¼ in. to provide suitable restraint to the lateral move-

Fig. 1—Remolding apparatus

ment of the concrete during the remolding operation. The apparatus is clamped to the horizontal surface of a vibrating table.

The concrete for the test is consolidated by the standard rodding procedure* in a mold, of standard slump cone dimensions, which is placed in the container, as shown in Fig. 1 and 2 a. Upon completion of consolidation, the mold is removed, the slump measured (Fig. 2b), if desired, and the rider assembly installed as shown in Fig. 2c.

Vibration is induced by the rotation of eccentrically mounted weights on a motor-driven shaft attached to the underside of the table surface. The switch which actuates the vibrating mechanism also starts an electric timing device which is graduated in tenths of seconds. Vibration is continued until the cone of concrete has been remolded to a flat disc. This condition is indicated by a graduation on the vertical rod attached to the rider. At the instant the end point is reached, the switch is turned off and the timing device is stopped; indicating the time required to remold the concrete from a cone to a disc (Fig. 2d).

*If the slump is expected to be less than 1½ in., the concrete may be consolidated by vibration in lieu of rodding. Vibration should continue 20 to 30 sec.

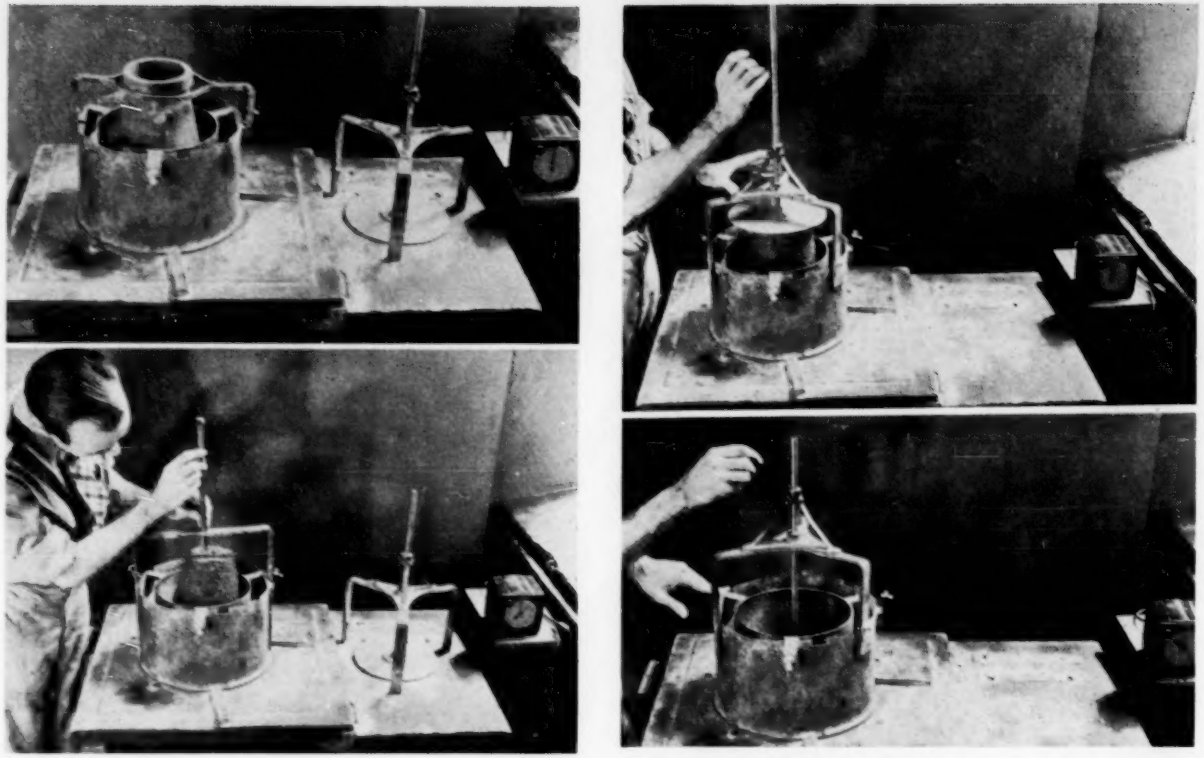

Fig. 2 (a, upper left)—Remolding apparatus on vibrating table ready to receive concrete; (b, lower left)—Measuring slump after removal of cone; (c, upper right)—Rider in place just prior to commencement of remolding. Pencil indicates end point to which remolding will be accomplished; (d, lower right)—Remolding completed. Pencil indicates end point. Clock at right indicates time of remolding (13.8 seconds)

The purpose of the inner ring is to cause the concrete to move downward and laterally by means of plastic flow rather than by crumbling and uncontrolled semi-plastic flow. Varying the clearance between the lower edge of the ring and the container bottom permits some simulation of field conditions of restraint, that is; the minimum clearance of 2½ in., would be used to measure the relative remoldability of concrete containing small maximum size coarse aggregate (1 in. or smaller) which is to be placed in thin heavily reinforced sections, and the 3¼ in., clearance would be used for concrete containing aggregate up to 2 in., in maximum size to be placed in relatively open and sparsely reinforced sections.

Vibration is used because vibratory impulses are more similar to field conditions than are the vertical drops of a flow table.

The vibratory remolding apparatus is sensitive to small differences in gradation and shape of the aggregates and of the inherent plasticity of the cement-water paste. With properly developed technique of operation, it provides a reliable index of the "remoldability" of mixtures of the drier consistencies. Mixtures of more fluid consistency than 2½ in. or 3 in., will segregate badly in the apparatus as they are likely to in the field where vibratory methods of placement are used.

The remolding apparatus is most valuable in the determination of the optimum amount of fine aggregate to be used with any particular combination of materials.

In this laboratory, the design of the mixture is based upon the water-cement ratio considered necessary to develop the degree of durability and strength in the structure. The shape and nature of the member, the facility with which placement of concrete may be accomplished and the means of consolidation proposed determine the consistency to be used. Therefore, for any structure and combination of materials, the water-cement ratio and consistency are predetermined factors. With these two fixed factors, the mixture is designed by the actual trial method, that is; the fine and coarse aggregate, whatever may be their size, shape and gradation, are added to the cement-water slurry until the desired consistency is obtained. The number of these trial batches which may be necessary is a function of operational technique. However, when the desired slump is obtained with what appears to be approximately the desired workability, the cement content, mixture proportions and sand-aggregate ratio are calculated. Then the accuracy of the design is *checked* by the remolding apparatus, as follows: (a) the "remolding time" of the trial design is determined; (b) the proportions of the aggregates in the mixture are recalculated, without changing the cement content or the water-cement ratio, and the "remolding time" of the mixtures, having sand-aggregate ratios above and below that of the trial design, are deter-

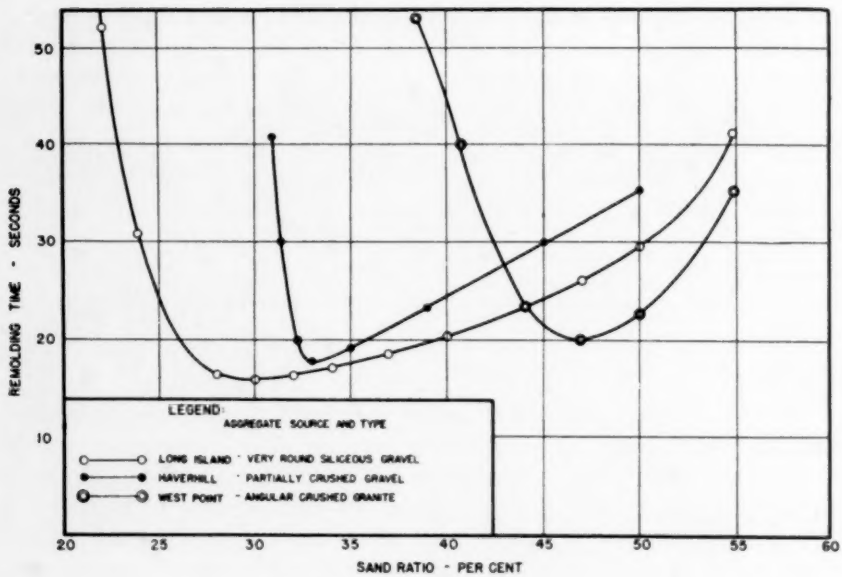

Fig. 3—Typical remolding curves

mined and plotted against sand-aggregate ratio. The low point on the curve which connects the points represents the optimum combination of the materials in the mixture for the particular use contemplated.

A group of remolding curves developed in designing mixtures from three different types of coarse aggregate is shown in Fig. 3. The mixtures possessed the same water-cement ratio, slump and fine aggregate, and the maximum size and gradation of the coarse aggregates were the same. The significant variable was the shape of the coarse aggregate and a difference in cement content as affected by that shape. The curve which commences near the upper left corner of Fig. 3 represents mixtures containing well-rounded siliceous gravel. The first mixture attempted contained 22 per cent (of the total aggregate present in the mix, by weight) sand and required 52 seconds to remold. An increase of 2 per cent in the sand ratio decreased the time required to remold to 31 seconds. Further increases in the sand ratio finally reduced the remolding time to 16 seconds at 30 per cent sand. Thus the time required to remold these ingredients, in a plastic mixture, was reduced to one-third by increasing the sand ratio by 8 per cent. Increasing the sand beyond the 30 per cent ratio, caused a gradual upturn in the curve, indicating loss in placing efficiency. In this case, the 30 per cent ratio is the optimum.

The curve at the right of Fig. 3 contains angular crushed granite having a large percentage of flat and elongated particles. The first mixture

attempted contained a sand ratio of 38 per cent and required a remolding time of 53 seconds. As the sand ratio was increased, the remoldability improved until the optimum was reached at 47 per cent sand. Increasing the sand beyond this value brought about a lessening of the remolding efficiency.

The condition of the concrete at the end of the remolding period may be examined for evidence of water-, sand- or other types of segregation. A properly designed mixture will possess a uniform distribution of materials throughout the mass after the remolding is completed.

A detailed description of the method of design of mixtures and use of the remolding apparatus is contained in the Handbook for Concrete and Cement distributed by this laboratory.

The remolding apparatus has been used in this laboratory for the design of concrete mixtures for a very large number of structures since 1937. Its effectiveness has been well demonstrated by the success of the designs when used in the field. Its use is limited definitely to mixtures of stiff consistency and small (2-in. maximum) size coarse aggregate. Careful and uniform operational technique is important in obtaining reproducible results.

The apparatus should be well suited for use by the manufacturers of concrete products where the molding of stiff concrete in thin sections by vibration is desired.

Powers' article, previously referred to, is recommended as an excellent source for a more complete treatment on the advantages, uses and limitations of the remolding apparatus in the design of concrete mixtures.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Cement grout effectively ends maintenance problem

HARRY F. SMITH, *Engineering News-Record*, V. 130, No. 22, (June 3, 1943), p. 22.
Reviewed by S. J. CHAMBERLIN

Railroad ballast is grouted where concrete driveways and storage platforms are constructed over and around track areas. The entire depth of 18-in. of 1½ to 2½ in. graded ballast is grouted and a 7-in. concrete slab constructed on top. The grout is distributed by chutes from ready-mix trucks and is worked in place by coarse fiber brooms.

Dollar economies in concrete proportioning

R. F. MOSS, *Engineering News-Record*, V. 130, No. 24 (June 17, 1943) pp. 94-96.
Reviewed by S. J. CHAMBERLIN

Modern precision control methods, supplemented by care in the selection and grading of aggregates and in the operations of placing and curing, can reduce the "factor of ignorance" in designing concrete mixtures. Examples are given in dollars and cents of the savings possible by accurate water control. Besides savings in cement, greater uniformity and durability may be obtained by proper field control.

Deep footings built without forms

H. A. SAURBREY, *Engineering News-Record*, V. 131, No. 3 (July 15, 1943), pp. 72-73.
Reviewed by S. J. CHAMBERLIN

Several hundred footings about 10 ft. deep were constructed in fine dense clay without formwork. A simple template of 3 x 12-in. timbers was laid on the surface and workmen with ordinary hand spades cut the outside perimeter of the holes, throwing the loosened clay to the middle where it was removed, together with the bulk of the excavation, with a clamshell bucket.

Precast box conduits for runway drainage

C. R. OTTO, *Engineering News-Record*, V. 130, No. 24 (June 17, 1943) pp. 87-88.
Reviewed by S. J. CHAMBERLIN

Dish-topped box conduits of interlocking two-piece precast concrete units have their tops slotted to receive the runoff and are set flush with the top of the runway to serve as an extension of the pavement. The bottom trough section and the cover slab with short side walls are 5 ft. long and provide inside areas of 10 x 10-in. and 15 x 15-in.

The units have 4-in. walls, a top thickness of 7½ in. and are unreinforced. Storm sewers are connected through single deep units acting as catchbasins.

Highway over dam rebuilt in winter

Engineering News-Record, V. 130, No. 18 (May 6, 1943), pp. 112-114. Reviewed by S. J. CHAMBERLIN

An old 10-ft. highway bridge was replaced by a 20-ft. concrete bridge in temperatures as low as 35 deg. below zero. A falsework was erected under the existing deck to provide a working platform and to support the forms and concrete of the new deck as well as a weather shelter overhead. The freshly-placed concrete of high-early strength cement was maintained at a temperature above 50 deg. for three days and then cooled at a rate not exceeding 20 deg. per day.

The arch and other theories as applied to portal frames

S. A. LOVELAND: *The Structural Engineer*, V. 21, No. 3 (Mar. 1943) pp. 83-114.
Reviewed by GEORGE C. ERNST

This paper contains Parts I and II of a rather comprehensive presentation of methods of elastic analysis of arches, portal frames, and framed bents. Virtual work and strain energy methods are discussed and developed in this issue; with slope deflection, neutral point, column analogy, and moment distribution planned for the next portion. Although nothing new is developed, the paper provides a condensed but reasonably complete assemblage of rigid frame theory.

Multiple-box drain for airbase runoff

Engineering News-Record, V. 130, No. 18 (May 6, 1943), pp. 108-111 Reviewed by S. J. CHAMBERLIN

The drain consists of 224 ft. of single box, 1,438 ft. of double box and 3,391 ft. of triple box. All of the passages are 5½ x 6-ft. except near the end. The bottom is 10½-in. thick, the exterior walls 8-in., the intermediate walls 6 in., and the roof 9 in. The boxes are built in alternate 75-ft. monoliths in two placements, first the bottom and 7-in. high sections of the walls and after placement of interior forms the sides and roof. Floating forms of the all-bolted type are used. Mixing is by a paver and the concrete is delivered by a crawler crane handling a 1-cu. yd. bottom-dump bucket.

Old concrete slab salvaged for new road

C. A. ROTHROCK, *Engineering News-Record*, V. 130, No. 18 (May 6, 1943), pp. 101-104.
Reviewed by S. J. CHAMBERLIN

The old two-lane road was utilized in constructing a dual-lane divided highway by placing a new 6-in. non-reinforced slab directly on the old concrete after the surface had been treated with asphalt and blast furnace slag screenings to eliminate bond between the old and new construction. In areas beyond the old slab an 8-in. non-reinforced pavement was built on a 4-in. compacted fill of blast furnace slag. Catch basins are located back of the curb where they can be protected by a concrete cover requiring little reinforcing steel.

Form work on outfall and main sewers in Sydney

Concrete and Constructional Engineering, V. 38, No. 5 (May, 1943) pp. 156-158.
Reviewed by GLENN MURPHY

A description of the construction features of a new sewerage submain for Sydney, New South Wales. Concreting the open cut was done out in four stages: invert to within two inches of the top, walls up to a height of four feet, remainder of walls and roof, and finally the top of the invert. Both $\frac{3}{16}$-in. sheet steel and plywood forms were used. In

the tunnel sections, space was limited and concrete had to be hand placed. Air and water bubbles on the face of the finished work were avoided by exclusion of surplus water from the surface of the water of the concrete by means of a vacuum pump and also by the use of an absorbent facing on the form.

Many-sided reservoir simplifies formwork

Engineering News-Record, V. 130, No. 24 (June 17, 1943) pp. 92-93. Reviewed by S. J. CHAMBERLIN

Two 140-ft. "diameter" reinforced-concrete water reservoirs were built with 46 sides, each about 10 ft. long, rather than circular to simplify the formwork and to permit reuse of the forms for other purposes. The open-top reservoirs have an 8-in. bottom slab and a wall thickness varying from 2 ft. 4 in. at the bottom to 1 ft. 3 in. at the 12-ft. level where they have a constant thickness for the remaining 12 ft. The variable thickness was placed in one casting for the entire circumference and the remaining height in another casting, resulting in one horizontal joint. The concrete was delivered by agitator truck, lifted by 1-yd. bottom-dump bucket handled by a crawler crane to a 3-cu. yd. hopper, and transported to forms by buggies.

Thin concrete roof withstands fire test

J. E. KALINKA, *Engineering News-Record*, V. 130, No. 22 (June 3, 1943), pp. 108-109.
Reviewed by S. J. CHAMBERLIN

The concrete of the Z-D barrel-shell roof, 3½ to 4½ in. thick on a 60 ft. span, was not fully hardened (due to cold weather) when a 75-ft. wide section of plywood forms and timber centering completely burned. For 50 to 75 ft. in either direction from the formed area, minor cracking and spalling occurred, while practically no visible damage occurred in the area where forms were still in place. A full-size loading test on one panel before repairs had been made indicated the concrete to be structurally sound. All damaged concrete was removed before grout and shotcrete were applied.

Repairs save concrete tank after settlement causes cracks

Engineering News-Record, V. 130, No. 20 (May 20, 1943), pp. 86-87. Reviewed by S. J. CHAMBERLIN

A three-compartment Imhoff tank sewage disposal tank cracked badly after filling. Sidewalls, designed as being simply supported at the top and fixed at the base, evidently failed as simply supported beams due to settlement at the center of the tank. A stiffening beam was designed for the bottom of the sidewalls consisting of two separate portions, one inside and one outside, with a minimum thickness of 8-in. and of varying depth. On the outside of the wall a mat of steel was extended from the top of the stiffening beam to the top of the tank and covered with shotcrete. On the inside of each of the compartments a basket of reinforced concrete 8 in. thick was shotcreted in place.

Building an 82-acre concrete roof with traveling retractable forms

Engineering News-Record, V. 130, No. 24 (June 17, 1943), pp. 62-65. Reviewed by S. J. CHAMBERLIN

The roof is a multiple barrel-arch 3 in. thick with 38-ft. spans. Deck and rib forms were built into an all-timber carriage, one arch wide and 4 bays (120 ft.) long, which moved on 56-in. artillery wheels on an 18-ft. gage. The plywood deck forms were dropped 4½ ft. to pass under the ribs and then raised by chain hoists to position. A work platform about 5-ft. below the rib-soffit level facilitated form erection. The entire frame was supported by screw-jacks and blocking during concrete operations and curing. One tower hoist served the concrete placing for a four-bay width, moving

ahead with the form. Over 100,000 cu. yd. of concrete was placed in six months during the fall and winter.

Concrete and timber aid fast war plant construction

Engineering News-Record, V. 130, No. 18 (May 6, 1943), pp. 96-100. Reviewed by S. J. CHAMBERLIN

The building, measuring 300 x 800 ft., has concrete floors, walls and columns. Carrying beams and roof beams are of laminated timber. Column supports in the non-basement area are continuous longitudinal footings often 9 ft. wide at the base. In the same way, the walls of the tunnels and corridors are continuous massive non-reinforced designs frequently of 12-ft. 8-in. base width and decreasing to a few feet at the top. For the basement area individual 9 x 9-ft. non-reinforced, stepped footings are used. Slabs placed on the ground are non-reinforced. Over the basement area is a 4-way flat slab 15-in. thick with 7-in. non-reinforced drop panels. The forms for the slab consisted of expanded steel bar trusses on 18-in. centers supporting a ¾-in. plywood deck. The exterior walls measure 49 ft. 4 in. to the top of the parapet and are reinforced with ½-in. round bars on 12-in. centers each way.

A reinforced concrete grain silo

OSCAR FABER, *Concrete and Constructional Engineering*, V. 38, No. 5 (May, 1943) pp. 147-155. Reviewed by GLENN MURPHY

The author describes the construction of a grain silo consisting of fifteen bins, each 8 ft. square and approximately 45 ft. high. The bins are arranged in three rows of five bins each. Reinforced concrete construction was used throughout—the mixture being 112 lb. of cement to two cubic feet of sand and four cubic feet of ¾-in. gravel. The principal construction feature was the use of sliding forms. One lift of forms was provided for the whole of the internal and external surfaces of all the bins. An hour or two after the completion of one lift, the entire form was raised by means of hydraulic jacks. One end of the reinforcement which was used at the juncture of the floor and columns was cast in the columns and the other end turned up along the columns until the walls were completed. The rods were then bent down and the surface of the columns roughened to provide a bond for the floors.

Concentric-ring concrete tank for Topeka

Engineering News-Record, V. 131, No. 5 (July 29, 1943) pp. 68-71. Reviewed by S. J. CHAMBERLIN

The tank, 87-ft. in diameter and 25 ft., high rests on three concentric 8-in. ring walls and an 8-in. outer wall of fluted form strengthened by pilasters. The tower walls are 120 ft. in height from the ground level to the bottom of the elevated tank. Another tank of the same size is in the foundation and is composed of three inner concentric rings of 8-in. walls and an outside wall of 18-in. thickness, all of which rise 24 ft. from the 5-ft. foundation slab to ground level. A heavy slab forms the roof of the underground tank and also serves as a support for the tower walls. The inner walls are discontinued at the upper tank and rows of columns on the same circles carry rings of roof beams over which the flat slab roof is supported. Sliding forms were used and a work platform on the rising structure served as the supporting form for the horizontal slabs for tank bottom and cover.

Hydrated calcium aluminates

LANSING S. WELLS, W. F. CLARKE and H. F. MCMURDIE, *Journal of Research*, National Bureau of Standards, May 1943, R. P. 1539. Reviewed by F. B. HORNIBROOK

The solid phases of the system CaO-Al_2O_3-H_2O were investigated by petrographical and X-ray diffraction methods. By means of the diffraction patterns, it was found that

the so-called hexagonal tricalcium aluminate hydrate is in reality a mixture of hexagonal $2CaO \cdot Al_2O_3 \cdot 8H_2O$ and hexagonal $4CaO \cdot Al_2O_3 \cdot 13H_2O$ intercrystallized in equimolecular proportions. On standing, dry dicalcium aluminate hydrate is slowly converted into the tetracalcium aluminate hydrate and hydrated alumina.

The only stable solid phases that occur in the system CaO-Al_2O_3-H_2O over the temperature range from 21 to 90 C are: Gibbsite, $Al_2O_3 \cdot 3H_2O$; the isometric tricalcium aluminate hexahydrate, $3CaO \cdot Al_2O_3$-$6H_2O$; and $Ca(OH)_2$.

The solubility curves of gibbsite and isometric $3CaO \cdot Al_2O_3 \cdot 6H_2O$ in the system CaO-Al_2O_3-H_2O were determined at 21 and 90 C.

Expansion of concrete due to reaction between andesitic aggregate and cement

H. A. Coombs: *Am. J. Sci.* 1942, 240 (4), 288-97. *Road Abstracts*, 1943, V. 10, No. 3, England. Highway Research Abstracts

After a brief review of the literature on the failure of concrete due to reaction between aggregate and high-alkali cement, an account is given of further investigations on concrete cores and test cylinders. The main conclusions are as follows: (1) In every case reported of excessive expansion of concrete caused by reaction between aggregate and high-alkali cement, the result of the reaction is a siliceous gel, thought to be formed by reaction between the alkalis and the more soluble silicates in the aggregate. (2) Reaction rims about the troublesome pebbles are detected microscopically by means of ultra-violet light and a new selective staining method. (3) Excessive expansion in concrete due to reaction between aggregate and cement is aggravated by cements containing inordinately high percentages of alkali. A lower alkali content (under 0.6 per cent) lessens the expansion markedly. Some easy way must be devised of detecting the aggregate liable to reaction before it is used in the concrete.

Douglas Dam constructed in 13 months

Engineering News-Record, V. 131, No. 3 (July 15, 1943), pp. 64-68. Reviewed by S. J. Chamberlin

Rain and leakage caused the greatest construction difficulties. Modified type "B" cement was used with a water-cement ratio of about 0.75 (wt.) and a cement content of 0.85 bbl. for the "mass" concrete. "Face" concrete had a water-cement ratio of 0.55 and a cement content of 1.15 bbl. Fine and coarse aggregate were produced on the site from limestone. Parts of the placing trestle that were to be embedded in the dam were built of reinforced concrete. Concrete was normally placed in 5-ft. vertical lifts with 72-hr. time intervals. A cooling system composed of small pipes carrying river water were used in strategic places to allow 10-ft. lifts with 48-hr. intervals. Closure was made by dropping specially designed reinforced concrete gates into vertical slots, made by steel beams previously embedded upright along the upstream face of the dam.

P. R. C. (Pre-stressed Reinforced Concrete)

Kurt Billig. 248 pp., cloth bound, 5½ x 8½, F. Billig, Jamaica, New York, $5.00. Reviewed by A. J. Boase

Part I, 58 pages, is essentially the same as P.R.C. Pre-stressed Reinforced Concrete, Part I, General Data, as reviewed in the Journal of the American Concrete Institute, February, 1943. Part II has now been added and the two appear under one binding.

In the second part the author develops a general mathematical theory and formulas for the design of pre-stressed reinforced concrete. A number of problems are worked out in detail to show the application of the author's equations and a number of tables are given which were designed to facilitate the calculations. A short bibliography is appended.

Unfortunately, the author has chosen a foreign nomenclature, which makes study of this book very difficult—in addition, it has an errata list of some 10 or 12 pages and this further complicates the study of this book.

Slip construction basins of thin concrete

Engineering News-Record, V. 131, No. 3 (July 15, 1943), pp. 82-85. Reviewed by S. J. CHAMBERLIN

The ship construction basins, 1200 x 82 ft., have 8-in. thick moderately reinforced floors and wall thicknesses varying from 8-in. at the bottom to 5-in. at the top. Counterforts reinforce the thin walls. Buoyancy of the closed basins, with floor 20 ft. below mean low tide of an adjacent 30-ft. deep channel, is resisted by a system of well points that provides for intermittent automatic pumping. Under the main section of the hull, the forms for the ship rest directly on a prepared and shaped section of the floor and the load on the fore and aft frames is evenly distributed. Bottom of hull is "stripped" by floating the ship. The concrete floor was placed directly on the undisturbed material at the bottom. A second placement was made of the "altar" section along each side of the dock and then the forms were erected for the 21-ft. high walls. The gate at the water end of the basin is a 65 x 28-ft. floatable concrete box cast in place. Concrete was designed for 4,000 psi with 6½ bags of cement, white silica sand and limestone of ¾-in. maximum size.

Some examples of reinforced concrete bridges having adjustable pre-stressed reinforcement

H. LOSSIER: *Genie Civil*, 1942, 119 (21), 253-7; (22), 269. *Building Science Abstracts*, V. 16, (New Series), No. 3, Mar. 1943, p. 47. HIGHWAY RESEARCH ABSTRACTS

Design and constructional details are given of some reinforced concrete bridges in which is used auxiliary reinforcement (uncovered but protected from weathering influences) capable of being automatically pre-stressed, the degree of pre-stressing being adjusted periodically according to requirements. By this means the unfavorable and variable effects due to prolonged shrinkage and creep in the concrete may, at any rate during the first few years, be controlled. Among the bridges described are: (1) the Aue Bridge, Germany (cantilever with a central span of 226.3 ft.); (2) the Risorgimenti Bridge across the Tiber in Rome (328 ft. span); (3) a bridge of 3 arches, each of 590.4 ft. span and of low rise. The particular application of adjustable pre-stressed reinforcement to the construction of long span fixed arches having isostatic horizontal thrust and with heights as low as 1/20 of the span is discussed. The use of "expanding cements" for putting the reinforcement in tension is also mentioned.

Highway Research Board proceedings

Papers and Reports presented at 22nd annual meeting, December 1942, approximately 500 pages.

The contents of V. 22 (1942) Proceedings, Highway Research Board include papers on economics, design, materials and construction maintenance, traffic, soils. On design are the following: War Department Building Road Network, Joseph Barnett; Runoff from Flight Strips, C. F. Izzard; Flexible Pavement Foundations, O. J. Porter; Soil Tests for Design of Runways, T. A. Middlebrooks and G. E. Bertram; Wheel Load Limits and Design, Keith Boyd; Flexible Pavement Design, M. G. Spangler; Variables in Concrete Runway Design, F. M. Baron, and Roadside Development, H. J. Neale.

On materials and construction: Effects of Calcium Chloride on Strength of Low Water Concrete, C. R. Cornthwaite and J. C. Yates; Volume Changes in Sand-Gravel Concrete, F. H. Jackson and W. F. Kellerman; Effect of Calcium Chloride on Dynamic

Modulus of Concrete, R. C. Valore and J. C. Yates, and Ohio Experience with Vibration of Concrete, C. W. Allen and J. F. Barbee.

On maintenance: Maintenance Equipment, T. H. Dennis, and Maintenance Costs, H. K. Bishop.

Effects of the addition of calcium chloride to portland cements and concretes

A. J. NEWMAN, *Concrete and Constructional Engineering*, V. 38, No. 5 (May, 1943) pp. 159-167.
Reviewed by GLENN MURPHY

A report of a comprehensive series of tests made by the Building Research Station. Two normal cements and two rapid-hardening cements were used with sand and gravel aggregates. The mixture was 1:2:4 by weight with a water-cement ratio of 0.60. Both granulated and flake $CaCl_2$ were used. Compression tests were made on 4-in. cubes. The following conclusions were noted: 1) The time of set of the cement was decreased by the addition of increasing amounts of $CaCl_2$. 2) At normal temperatures, the addition of $CaCl_2$ accelerates the rate of strength gain at early ages. 3) Addition of $CaCl_2$ increased the strength of air-stored concrete for ages up to three months. For ages over three days, more than a 2 per cent $CaCl_2$ did not cause any additional increase in the strength. 4) Workability of the concrete was increased by the addition of up to 4 per cent $CaCl_2$. 5) The shrinkage of the concrete was increased by additions of $CaCl_2$ 6) The addition of 2 per cent $CaCl_2$ results in a slight corrosion of steel reinforcement. 7) Addition of $CaCl_2$ increases the strength of concrete stored at low temperatures but does not add protection against freezing when the temperature is continuously below 32 F.

Nature of the prismatic dark interstitial material in portland cement clinker

WILLIAM C. TAYLOR: *Journal of Research*, National Bureau of Standards, *30* (1943) R.P. 1536.
AUTHOR'S ABSTRACT

As a result of a series of previous investigations by the phase equilibrium method, it has been concluded that K_2SO_4 and K_2O . 23CaO . $12SiO_2$ are the only compounds containing K_2O that may exist when a mixture of K_2O, CaO, MgO, Al_2O_3, Fe_2O_3, SiO_2, and SO_3, in the proportions occurring in portland cement clinker, is heated and cooled under equilibrium conditions. Frequent observations of a prismatic dark interstitial phase in etched, polished sections of commercial clinker and in rapidly cooled laboratory clinkers led to the present study of the relation of this phase to the system K_2O . 23CaO . $12SiO_2$-CaO-5CaO-5CaO . $3Al_2O_3$ and to other systems involving K_2O. The prismatic phase has not been found to exist under equilibrium conditions, but has been produced only under conditions of rapid cooling. Both thermal and compositional conditions that have been found to be required for its formation suggest that it may be a metastable form of 3CaO . Al_2O_3 and monotropic with respect to the stable, isotropic, rectangular 3CaO . Al_2O_3. That portion of K_2O combined as K_2SO_4 has been found to have no effect in inducing the formation of the prismatic phase. The conditions necessary for the formation of this phase have been outlined. Although the exact composition has not been established definitely, because of the experimental difficulties that have been outlined, the close relationship between the prismatic phase and 3CaO . Al_2O_3 leads to the recommendation that this phase be referred to as "prismatic 3CaO . Al_2O_3" instead of the term "prismatic dark interstitial material." Photomicrographs are presented.

Ten year tests on commercial masonry cements

R. L. BLAINE, *Journal of Research*, National Bureau of Standards, July, 1943, Research Paper 1548.
AUTHOR'S ABSTRACT

In an investigation of the properties of 41 commercial masonry cements reported in 1934 additional specimens were made for tests at a later age. Two-inch mortar cubes

made of these cements were tested in compression after both water and air storage for 10 years. Mortar bars specimens were measured for linear change after storage in water for 10 years.

The compressive strengths of the 1 to 3, cement to standard sand mortar cubes varied from 200 to 6000 psi at 10 years. The mortars made of cements originally classified as largely portland, portland plus unknown additions, and portland cement-hydrated lime mixes, had greater compressive strength at 10 years after air storage than after water storage. The mortars made of these types of cements had very little increase or a slight decrease in compressive strength between the periods of 1 and 10 years of water storage, but had appreciable strength increases in air storage for the same period. Mortars made of cements classified as natural cements, and largely slag cements had greater compressive strengths after water storage than after air storage. The mortars made of the natural cements increased only slightly in compressive strength while those made of the slag cements decreased slightly between the 1- and 10-year periods of air storage.

The linear expansion of the mortar specimens varied from 0 to 0.808 per cent. Cements of greater magnesia contents showed a tendency towards greater expansion. All the mortar specimens made of cements having less than 4 per cent magnesia had linear expansions of less than 0.04 per cent in 10 years of water storage.

Sea-going barges of concrete construction

Engineering News-Record, V. 130, No. 24 (June 17, 1943), pp. 66-71. Reviewed by S. J. CHAMBERLIN

Concrete barges of the conventional shape of a ship are being placed continuously almost to deck height. Concrete in the bottom is 5-in. thick, which covers two layers of 1-in. sq. longitudinal bars with a ½-in. round spacer bar between and ½-in. round transverse bars top and bottom. Side shells are 4¼-in. thick and are strengthened by longitudinal concrete beams and bulkheads. Eighty per cent of the reinforcement and practically all forms except the deck are in place before concreting starts. Longitudinal steel in bottom and deck is butt-welded continuous for the full length. The exterior forms consist of pre-assembled timber bents which support plywood a few inches in front of the frame itself. At the bilge a series of sawcuts in the reverse side of the plywood permit bending. Plywood forms for the interior are built on a wood structure in the carpenter yard on which the exterior lines and the interior framing of the ship are laid out. Snap-ties for taking the thrust of the forms are placed in the 18-in. thick beam sections rather than in the thin sections of the walls. Fine and coarse aggregate (⅜ in. max.) of hard-burned clay with 9¼ bags of cement per cu. yd. and 5¼ gal. of water per sack produce a 5,700 psi concrete weighing 108 lb. per cu. ft. A powdered admixture and some natural silica sand are added to increase the workability. Internal and external vibrators are used on the 5 to 6-in. slump concrete. The hulls are tested for watertightness by applying hydrostatic head. Testing and remedy of minor leakage is more time consuming than repair of the obvious defects of the concrete. Interior surfaces of the oil compartments are treated with three coats of a special paint designed to protect the concrete from oil. The exterior to the water-line is given a coat of paint intended to neutralize the free lime and to seal the pores and is followed by two coats of low visibility gray.

Function of carbon dioxide in producing efflorescence on plaster and cement products

DANA L. BISHOP, *Jl. of Research*, (National Bureau of Standards) May 1943; pp. 361-6.
AUTHOR'S REVIEW

Efflorescence occurs as a deposit of crystalline salts on the surface of plaster or masonry. The salts ordinarily are present in the interior of the plaster or masonry and are dis-

solved by water, transported to the surface and there deposited by evaporation of the water.

Magnesium sulfate although not a constituent of plaster white coat or base coat, was found to be a constituent of efflorescence. Its formation was accounted for by the reaction of water and CO_2 with the white-coat plaster. Carbon dioxide converts the magnesia to slightly soluble $MgCO_3 . 3H_2O$ which reacts with gypsum forming $MgSO_4$. Magnesium sulfate was produced experimentally by the action of CO_2 on a water suspension of MgO and $CaSO_4 2H_2O$. Hydrated magnesium sulfate efflorescence also was formed on white-coat plaster specimens containing dolomitic hydrated lime and gypsum when treated with moist CO_2 and then allowed to dry.

$MgSO_4$ also is found as efflorescence on cement products, possibly being derived from MgO and sulfate in the cement by reaction with CO_2 and water. The compound $3CaO . Al_2O_3 . 3CaSO_4 . 31H_2O$, a constituent of cement was decomposed by CO_2 and water, making $CaSO_4$ available for reaction with $MgCO_3 . 3H_2O$ to form soluble $MgSO_4$.

Analysis of data published by Rogers and Blaine (J. Research NBS *13* 811, (1934) RP 746) on 41 commercial masonry cements showed that mortars which produced no efflorescence were made from cements having an average MgO content of 5.0 per cent, those showing slight efflorescence 8.8 per cent and those showing pronounced efflorescence 19.7 per cent. The amount of $MgSO_4$ formed in cement products would be limited by the amount of sulfate or magnesia present. Magnesia might be extracted also by water and CO_2 from magnesium carbonate in the sand or aggregate. Magnesium chloride also might be formed from reaction of $CaCl_2$ with $MgCO_3 . 3H_2O$.

Effects of aggregate characteristics on durability of concrete

F. F. BARTEL: (*Thesis submitted to the Faculty of the Graduate School of the University of Maryland in partial fulfillment of the requirements for the degree of Master of Science.*) Washington, D. C., 1942 (National Sand and Gravel Association). From BUILDING SCIENCE ABSTRACTS

The investigation described in this thesis presents information helpful in evaluating the effects of quality of aggregate on the durability of concrete. The principal tests consisted of accelerated freezing and thawing of concrete made with different aggregates, the effects being measured by the variation in the modulus of elasticity of the concrete as determined dynamically. Particular attention was given to gravels of chert origin, to the effect of moisture in the coarse aggregate, and to the availability of moisture to both concrete and aggregate during the freezing and thawing process. Other points studied were the ability of deteriorated concrete to recuperate, the environment during exposure, and the specific gravity of the gravel. Exploratory tests were made to determine whether durability could be improved by using rich mixes, special cements, or certain admixtures. Twenty-nine coarse aggregates and eight sands, with widely differing mineral compositions and other physical characteristics, were tested. Resistance to freezing and thawing in concrete was compared with performance in the field and with conventional tests of aggregates and concrete. An important finding was that the presence of moisture in aggregate and concrete greatly lowered the durability of concrete made from chert gravels. The performance of concrete made from such material may therefore be improved by prior removal of some or all of the moisture from the gravel and by thorough drying out and/or drainage of the concrete before exposure. Fair correlation was found between service record and freezing and thawing tests but no agreement with conventional soundness tests. The service record of concretes losing less than 50 per cent of their modulus of elasticity in 50 cycles of freezing and thawing was generally favorable. Resistance to freezing and thawing was roughly a function of the percentage of deleterious particles, such as white porous chert, argillaceous limestone or sandstone.

Composition and physical properties of aqueous extracts from portland cement clinker pastes containing added materials

GEORGE L. KALOUSEK, C. H. JUMPER, and J. J. TREGONING, *Journal of Research*, National Bureau of Standards, 30, 215 (1943) RP 1530. Reviewed by F. B. HORNIBROOK

Mixtures of twelve commercial portland cement clinkers and water were filtered at 7 minutes and at 2 hours after mixing and the chemical composition, pH, conductivity surface tension, and density of the extracts were determined. The effects on the composition and physical properties of the extracts, produced by the addition of small amounts of various materials to the pastes, were studied in parallel experiments. The added materials included gypsum, calcium chloride, calcium acetate, fluosilicic acid, sucrose, T D A, tannic acid and triethanolamine.

It was found that from 1 to 58 per cent of the total potash and from 0.4 to 30 per cent of the total soda were extracted in 7 minutes from the clinker pastes containing none of the added materials. In the interval between 7 minutes and 2 hours, the percentage increases in the amounts of potash ranged from 0 to 9, and from 1 to 12 for soda. Triethanolamine increased the extraction of the alkalies from the clinker pastes more than any other material and gypsum caused decreases which, for the most part, were small.

The amounts of silica and lime in the extracts corresponded approximately to their reported solubility relations in the system lime-silica-water at 30 C.

The amounts of alumina in solution were small except when organic materials had been added to the pastes. Ferric oxide was either absent or present only in small amounts except in filtrates containing triethanolamine.

The SO_3 usually decreased in concentration between 7 minutes and 2 hours, precipitating as a sulfoaluminate. Computations based on the amounts of calcium chloride that had combined with alumina at both 7 minutes and 2 hours indicated that only small amounts of alumina had reacted.

Sucrose accelerated the early reactions involving the alumina bearing constituents of the clinkers.

On the problem of the practical importance of thorough compaction of concrete

H. LENHARD: *Zement*, 1942, 31 (11/12), 123-30; (13/14), 143-9. From BUILDING SCIENCE ABSTRACTS

The author examines first briefly the water/cement ratio-strength relation of concrete, as developed by Abrams and considers the significance of the workability of the mix. The recent investigations of Glanville, Collins and Matthews on the influence of aggregate grading and the importance of thorough compaction of the concrete mix for strength are then reviewed. Investigation has been made also by the author on the compaction of concrete mixes containing variously graded aggregate and on the compressive strength obtained. An account of the author's experiment is given. From these experiments the author finds that all concrete mixes in which the grading of the aggregate is the only variable, can be equally fully compacted provided that sufficient energy is used. The proportion of voids in the aggregate is, therefore, the same in each of the mixes so prepared and corresponds to the quantity of cement-water per unit of volume. The compressive strength of the concrete from the thoroughly compacted mixes so prepared is practically the same. An influence due to aggregate grading exists, therefore, where compaction is incomplete and non-uniform, and it must be concluded that the problem of aggregate grading is only one of compaction of the mix. There is no most favorable grading in respect of packing but a given grading may be particularly satisfactory in respect of the workability of the mix. Where it is necessary to keep the mix consistence as nearly as possible constant in view of the compaction energy at disposal, the proportion of the cement-water mixture used with quite differently graded aggregate must be correspondingly altered while the water/cement ratio remains the same and the pro-

portion of aggregate is increased or reduced corresponding to the varying proportion of the cement + water. Complete compaction of the concrete is then possible and the concrete compressive strength is practically unchanged. If, however, the compaction energy can be varied, only a slight change, if any, in the proportion of cement + water may be necessary with variation of the concrete aggregate where the concrete compressive strength is to remain the same. In both cases, i.e. whether the compaction energy can be varied or not, the use of a particular aggregate grading is dependent only on its economy.

Effects of added materials on some properties of hydrating portland cement clinkers

EDWIN S. NEWMAN, RAYMOND L. BLAINE, CHAS. H. JUMPER and GEORGE L. KALOUSEK: *Journal of Research*, National Bureau of Standards, Vol. 30, April 1943, R.P. 1533.Reviewed by F. B. HORNIBROOK

Whereas R.P. 1530 describes properties of 12 clinker paste *extracts* with and without added materials, this paper, R.P. 1533, describes tests made on the clinker *pastes* of the same 12 clinkers, with and without the same group of additions. It was found that although some of the materials caused large changes in the behavior of the clinker pastes during the first few days after mixing, these effects had largely disappeared by 28 days.

Following is a tabulation of the principal trends observed in the changes caused by the addition of the various materials to each of the clinkers. Of course numerous individual exceptions occurred and are pointed out in the paper.

Temp. change	Gypsum	Calcium Chloride	Calcium Acetate	Sugar	Tannic Acid	Triethanolamine	T D A	Fluosilicic Acid
15 Min.	decrease	variable	variable	increase	increase	increase	increase	increase
Maximum	increase	increase	variable	no change	variable	increase	no change	decrease
Time to reach maximum	decrease	decrease	increase	increase	increase	decrease	no change	increase
Flow	increase	increase	variable	increase	no change	decrease	increase	small change
Heat of Hydration								
½ day	increase	increase	decrease	variable	decrease	increase	increase	decrease
1 day	increase	increase	decrease	variable	decrease	increase	small change	decrease
7 days	small change	variable	increase	small change	small change	increase	small change	increase
28 days	no change	variable	small increase	small change	small increase	small increase	small change	small increase
Strength of Pebble Concrete								
1 & 3 days age	increase	increase	variable	decrease	decrease	.....	variable	decrease
28 days	small increase	small increase	small increase	increase	small change	.....	small change	small change
Merriman Sugar Solubility Test (Change in clear pt.)	decrease	increase	small increase	.....	decrease	variable	small effect	decrease
Floc Test	..... Slight change only throughout							

Waterproofings for unit-masonry walls

CYRUS C. FISHBURN and DOUGLAS E. PARSONS. Building Materials and Structures Report BMS 93, National Bureau of Standards. (Dupt. of Documents, GPO, Washington, D. C.) AUTHORS' ABSTRACT

The effectiveness of cement-water paints and of other waterproofings for unit-masonry walls was determined by measuring the water permeability before and after treatment of small wall specimens that leaked when exposed to simulated service condititions. The durability of some of the treatments was measured by again testing the walls after they had been stored outdoors for 1 or 2 years.

Cement-water paints were found to be highly resistant to water penetration and were more effective than emulsified resin or oil base paints. The cement-water paint coatings were durable and proved effective as waterproofings after 1 or 2 years of exposure, although some of the specimens were so weather-stained that their repainting might have been considered desirable from the standpoint of appearance. On rough-textured concrete block walls, the cement-water paint coatings were most effective when applied with stiff, fender-cleaning brushes rather than with soft-bristled whitewash or paint brushes. The admixture of fine sand in the first coat of paint applied to walls of rough-textured units, such as cinder-concrete block, was highly effective. Such coatings were durable and contained few or no pin holes. Coatings made of cement-water paint of thin consistency were more permeable than those made from a paint of medium consistency, but heavy applications of a medium consistency paint were less durable than thinner coatings of the same paint. Cement-water paint coatings applied to dry walls were more permeable than similar coatings applied to backings that had been wetted and were damp when painted.

Colorless waterproofing materials were of little or no benefit as waterproofing when applied to walls that leaked badly. Of the colorless materials tested, only one was effective, and walls treated with it were highly permeable when again tested after they had been exposed outdoors. The only effective and durable waterproofing treatment for brick walls that did not change the appearance of the walls, was repointing or grouting of the face joints.

Bituminous coatings applied to the inside, unexposed, faces of the specimens were ineffective as waterproofings and were badly blistered after a test exposure lasting 1 day. Brush coatings of portlant cement and sand were more effective than bituminous coatings, but were not as effective as trowel coatings of cement and sand prepared with or without admixtures of powdered iron and salammoniac (iron waterproofing).

Experience in the maintenance of concrete roads

W. SCHONBERG: *Strassenbau*, 1942, 33 (7/8), 34-40. *Road Abstracts*, V. 10, No. 6.
HIGHWAY RESEARCH ABSTRACTS

In a paper delivered at a meeting of the Concrete Roads Group of the German Road Research Society (Forschungsgesselschaft fur das Strassenwesen), common maintenance requirements on concrete roads are discussed and, on the basis of experience gained through work on the German motor roads, suggestions are made for future design and construction. (1) *Joints.* Considerable maintenance is required on joints, owing largely to loss of elasticity by the joint filler, or to shrinkage of the wooden inlay, allowing joint filler to flow into the cracks so caused. In post-war construction the solution may be the use of rubber-based fillers. New types of dowel have been suggested, and a flexible waterproofing membrane embedded in the slab on each side of the joint might be advantageously used. Since dummy joints are easier to maintain than ordinary expansion joints, their use is advocated. By using two dummy joints between each two expansion joints the total length of expansion jointing per mile on the motor roads has been reduced to less than half. Edges of joints should be carefully finished. Longitudinal joints between carriageway slabs and the marginal strips should have tie-bars. (2) *Cracking.* A tabulated record is given of the cracking observed in 1939 and 1940 on an 87-mile stretch of road constructed 1936-38, and of the average strengths of cores cut from the different sections. Since the sections built in any one year showed approximately the same number of cracks, it is probable that frost-heaving or settlement played little part in causing cracks, and that the main causes were shrinkage or stresses either inherent in the concrete or due to the method of construction. Internal stresses can be corrected by the use of more reinforcement. Since more cracks appeared in sections

where all expansion joints were dowelled than in those where dowels were exceptional, displacement of dowels in the placing and compaction of the concrete is probably responsible for some cracking. Some cracking was associated with consolidation under traffic of the insulation course, which is difficult to compact during construction owing to the common use of uniformly graded sand. It is suggested that the grading of this material should in future be specified. (3) Surface irregularity may be caused by flowing of too wet a mix after placing, or by careless work in the marginal joint. Considerable irregularity may be caused by subsidence at approaches to bridges, etc. Measures are recommended that may be taken in building approaches to minimize this danger. The use of permanent built-in central traffic markers is advocated, and reference is made to the use of a resin admixture in cement to prevent the scaling caused by the use of chemicals in winter.

Properties of porous concrete

PERRY H. PETERSEN. Building Materials and Structures Report BMS96. National Bureau of Standards. Reviewed by F. B. HORNIBROOK

The physical properties of a type of porous concrete consisting solely of portland cement, water, and uniform size gravel are described. Each of three coarse aggregates, grits, pea gravel, and ¾ in. gravel was used, with 2½ bags of cement per cubic yard in concrete tamped in place, and 3 bags per cubic yard when no compacting was done. Walls, wallettes, beams and bond pull-out specimens were tested, as well as 6- by 12-in. control cylinders. Compressive, transverse, shearing, and bond strengths are reported, as well as resistance to heat transfer, water penetration by capillarity and rain penetration.

The weight per cubic foot tended to increase with increase in size of aggregate, with a range of 97 to 105 lb. per cu. ft. for that placed loose, and 110 to 115 lb. per cu. ft. for that tamped in place. The modulus of rupture for the monolithic walls was about 100 psi, tending to be slightly less than this value for the concrete placed loose and more for concrete placed by tamping.

The compressive strength of the tamped cylinders ranged from 600 to 900 psi, and was a fair indication of the strength in walls 8 ft. high and of the same mixture. The values for secant modulus of elasticity at a stress of 200 psi ranged between 1 and 2 million psi, tending to increase with increased size of aggregate.

Resistance to heat transmission decreased as the size of aggregate increased, the values of thermal transmittance U, varying from 0.64 to 0.87 BTU per hour per sq. ft. per deg. F for 6-in. walls.

Resistance to rain penetration of walls to which ⅛-in. stucco was applied was excellent, and the coefficient of thermal expansion averaged 0.000006 per deg. F.

The amount of shrinkage ranged from 0.009 to 0.052 per cent for the dry walls and seemed to decrease with increase in size of gravel and with the amount of compacting.

The rise of water by capillarity in walls of porous concrete ranged from 3 to 6½ in. at 7 days.

The bond stress at first slip and the maximum bond strength between reinforcing steel and tamped porous concrete using pea gravel and 2½ bags of cement per cubic yard were about three-fourths and one-third respectively of that obtained for a regular concrete with a compressive strength of 2,500 psi. The computed shearing stress in the beams at failure was about 80 psi, this value being about one-half of that expected for a 2,500 psi concrete.

Width and spacing of tensile cracks in axially reinforced concrete cylinders

DAVID WATSTEIN and DOUGLAS E. PARSONS, *Journal of Research*, National Bureau of Standards, *31*, 1, July 1943, RP1545 (obtainable from Superintendent of Documents, Government Printing Office, Washington, D. C., price 10¢.)

AUTHORS' ABSTRACT

This paper deals with the factors governing the spacing and width of cracks in symmetrically reinforced concrete members subjected to axial tension.

The specimens for measurement of width and spacing of cracks were cylinders 4 ft. long and 3½, 4, and 5 in. in diameter. Five types of reinforcement bars included ⅞-in. deformed, plain and threaded bars, ⅝-in. deformed bars used in pairs and a No. 8 webbed Isteg bar. The entire series of tests was made with concretes of two different mixes having nominal strengths of 3,000 and 5,000 psi at 28 days.

Widths of tensile cracks in specimens with equal percentages of reinforcement were least for cylinders reinforced with pairs of ⅝-in. deformed bars or with ⅞-in. threaded bars. For the width of crack observed in specimens reinforced with plain hot-rolled bars carrying a stress of 20,000 psi, the corresponding stresses for the other bars were, on the average, 24,000 psi with the webbed Isteg bars, 25,000 psi with the ⅞-in. deformed bars, and 35,000 psi with the threaded and the ⅝-in. deformed bars. There was no significant difference between the widths of cracks for a given stress and a given type of bar, as observed in specimens of "weak" and "strong" concrete. It was observed that widths of cracks in cylinders of the same diameter and for different bars, stressed to the same extent, varied linearly with the spacing of cracks.

The test data and the theoretical equations were in good agreement with respect to the effects of the principal factors controlling the spacing and width of cracks. The spacing and the width of cracks were found to depend chiefly on the ratio of the diameter to the percentage of reinforcement and the nature of the deformations on the bars.

The paper summarizes the results of previous investigations of the cracking of reinforced concrete in tension, and the results of other investigators are compared with the authors' test data.

The authors conclude that the width of cracks in symmetrically reinforced concrete members subjected to axial tension and to the effects of shrinkage of concrete and change in temperature may be minimized by: (a) Using concrete of low shrinkage properties; (b) Limiting tensile stresses to low values; (c) Reinforcing the concrete with the largest number of bars of the smallest sizes compatible with other requirements; (d) Using deformed bars having projecting lugs that afford resistance against slipping without causing splitting of the concrete and that are spaced to provide the bearing area necessary to develop the desired bond strength.

Introduction to highway engineering

JOHN H. BATEMAN, 4th Ed. 459 pages. Illustrated. John Wiley & Sons Inc. Reviewed by R. W. CRUM

This work is well planned to give a comprehensive picture of the practice of highway engineering, each chapter being in effect a synopsis of the subject matter that would most likely be included in a lengthy engineering treatise on the subject. Its purpose and scope are well presented in the author's preface.

"This book was written to serve as a textbook for students of civil engineering in an introductory course in highway engineering. Emphasis has been placed on fundamental principles and processes in the practice of highway engineering. It has been assumed that the student will receive instruction in basic courses in surveying, drafting, mechanics of materials, chemistry, materials of construction, and structural design, and therefore detailed descriptions of laboratory procedures, test methods, manufacture of

materials, the proportioning of concrete mixtures, surveying operations, and the design of bridges have been avoided.

"In the first chapter the student is given a brief historical background and concise descriptions of highway systems and the highway structure itself. The following five chapters deal with subjects which should precede the study of highway surfaces. Details of the design, construction, and maintenance of various highway types then are described. These subjects serve as a logical background for a discussion of surveys, plans, specifications, economics, planning, financing, and administration."

Perusal of this book should give a student a good idea of what it is all about.

Chapter headings are: Development of Highways; Highway Subgrade Soils; Drainage; General Features of Highway Design; Earthwork; Bituminous Materials; Theory of Structural Design of Flexible Pavements; Low Type Road Surfaces and Foundations; Broken Stone Surfaces and Foundations; Bituminous Surface Treatments and Low Cost Bituminous Surfaces; Theory of Structural Design of Concrete Pavements; Concrete Pavements and Foundations; Sheet Asphalt and Asphaltic Concrete Pavements; Rock Asphalt Pavements; Block Pavements Highway and Street Maintenance; Surveys, Plans, Specifications and Supervision of Construction; Economics of Highway Improvement; Highway Planning, Financing and Administration.

There are three useful appendices on: Determination of Street Intersection Elevations, Tables for Determining Volumes of Earthwork, and Problems.

The theory of structural design of concrete pavements is developed from consideration of the stresses to which the pavement is subjected; abrasive stresses caused by tires, compressive and shearing stresses caused by wheel loads, stresses resulting from deflections under wheel loads, stresses caused by expansion and contraction of the concrete and temperature stresses. The effects of expansion and contraction of concrete, and of warping caused by temperature differentials are discussed in relation to joint spacing. Obviously much literature was carefully reviewed for preparation of the section on design of thickness.

The chapter on concrete pavements and foundations covers design of cross sections, crowns, longitudinal and transverse joints, use of reinforcing, materials, construction methods and equipment, maintenance of pavements and bases.

The chapter on highway subgrade soils is important to comprehension of the effects of subgrade characteristics upon the design and behavior of concrete as well as other types of surfaces.

Each chapter is concluded with a bibliography, which although not exhaustive, gives a good selection for reference reading.

American Highway Practice, Vol. II

LAWRENCE ILSLEY HEWES, 492 p. illus., John Wiley & Sons, Inc. Reviewed by R. W. CRUM

Dr. Hewes has used with telling effect the old pedagogical principle that although you can't teach a student everything he should know, you can at least tell him where to find it. The wealth of information in the book together with the extensive sources disclosed by the bibliographic notes at the end of each chapter should lead the inquiring engineer to the current information on nearly every phase of highway practice. And, even more, highway development being still very much alive, knowledge of the sources of information listed should help one later to further significant writings yet to come.

In other words, this two volume treatise is a first class reference book on highway engineering.

The author is qualified by long experience as Chief of the Western Region of the Public Roads Administration to discuss authoritatively the design and construction of highway pavements. This he has done exhaustively ln Volume II in chapters on penetration macadam, sheet asphalt, asphaltic or bituminous concrete, portland cement concrete, design of cement concrete pavement mixtures, design of concrete road slabs, brick roads, and miscellaneous structures. Useful appendices are added on mechanical float, standard details for box culverts, theory of rigid-pipe culvert back fills, miscellaneous small drainage structure items and details for highway tunnels. A large part of the volume is devoted to concrete pavements and minor concrete highway structures.

A noteworthy feature of the work is the extensive supporting data with examples given in smaller type to aid the user in selective reading.

Stating that: "Although modern portland-cement pavement construction is today well standardized, it is nevertheless a highly technical process. Recent practice is the result of intense and prolonged laboratory and field experiment and research. Field mastery of every detail requires much patience and experience," the author, after a brief historical note, goes on to discuss in detail the general characteristics of concrete pavements, subgrades, forms, mixing and placing, joints and reinforcement, compacting and finishing, vibrating, curing, super-elevating and widening, costs, warping of slab, test specimens, and maintenance. Current practices are presented with their background and reasons for development. Many illustrative examples are given with excerpts from typical specifications of several large road building states and the American Association of State Highway Officials.

If the reader is subjected to some confusion due to the varying descriptions of practices in different places he should bear in mind that concrete pavement practice is by no means static and that practically all highway engineers are continually striving to devise improvements. Two things are indicated by this situation: (1) that many questions have not yet been fully answered and (2) that in many respects there are more ways than one of securing satisfactory results.

In the chapter on design of mixtures the influences of water-cement ration, grading of aggregates, workability of concrete, mortar voids and concrete voids are discussed with references and illustrations from the research of Abrams, Talbot and Richart and others. In a section on practical application of mix, design theories methods are given for the use of trial mixes in applying the various factors previously mentioned to the determination of batch relationships and cement factors. "A systematic and thorough method of handling the problem of determining the cement content for given tensile strength for several aggregates is illustrated by investigations made by the Public Roads Administration in 1936 on North Carolina materials with the cooperation of the State highway commission." Excerpts relating to proportioning are given from the standard specifications of the American Association of State Highway Officials and the state highway specifications of Nebraska, California, and Illinois.

Treatment of the design of concrete road slabs is no less thorough. Taking up transverse joints, longitudinal joints, corner breaks, temperature-warping stresses, wheel loads, interior-load stresses, edge-load stresses, cross section design, use of reinforcing and dowels in transverse joints current methods of design are well presented. Reviews of the experimental results secured on the Columbia Pike in Virginia, the test road at Pittsburgh, California and the Bates road in Illinois and of the writings of Older, Westergaard, Kelley, Spangler, Pauls, Teller and Sutherland and others, trace the development of design practices and furnish supporting evidences of their validity.